GEOTECHNICAL STUDY
TEST SECTIONS
ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

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Attention: Mr. Dan Heilman, P.E.

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Introduction

Fugro Consultants LP is pleased to present this final report of our geotechnical study for the above referenced project. Mr. Neil McLellan, P.E. with Shiner Moseley requested this study based on several discussions with Mr. John Juenger, P.E. of our staff. Several proposals were submitted to Shiner Moseley during March and early April of 2004 discussing scope of services for various alternatives of test sections to be built. We performed our services in general accordance with Fugro Proposal No. 0401-2000-1409-4 dated April 08, 2004. A draft of this report was submitted to Shiner Moseley on August 4, 2004 for their review and comments. This report addresses Shiner Moseley's comments and supersedes all previously provided information.

Project Description

We understand that coastal erosion claims about 25 to 35 square miles of existing land in Louisiana each year. It was estimated that erosion claims an average of 35 to 50 ft of the Rockefeller Refuge each year along its 9-mile western border. The Louisiana Department of Natural Resources has been working on combating erosion through the Coastal Wetlands Planning and Protection and Restoration Act. As part of their plan to combat erosion, the Louisiana DNR is planning to construct a shoreline stabilization structure from Joseph's Harbor westward about 10 miles to the west boundary of the Rockefeller Refuge along the existing beach. Shiner Moseley performed a feasibility study in order to identify the options that would provide needed protection to the Refuge. Several of these options along with an additional option developed subsequent to the completion of the feasibility study have been identified and will be taken forward to next phase of design. These options include: a rock reef breakwater with or without a lightweight aggregate core, a gravel beach nourishment program, a concrete panel breakwater, and preloading the soils along the proposed alignment to promote strength gain.

Reef breakwaters are low crested structures designed to reduce wave energy while allowing some wave transmission under storm conditions. In order to reduce the bearing pressure on the foundation soils due to the weight of the rock, the rock core of the structure may be replaced with lightweight aggregate (LWA). Reportedly, the breakwater will be constructed in the Gulf of Mexico just off the beach, or along the shoreline subsequent to preloading using dredge surcharge as noted below. A beach nourishment program is also being considered. This program would basically consist of placement of graded gravel along the beach to prevent the removal of soils, however not completely stop all wave action. The beach fill may be built behind the reef breakwater.

The concrete panel breakwater will consist of a series of concrete panels with pre-cast concrete caps supporting a sheet pile wall. Concrete piles will be driven into the stiff clays existing below about 40 ft. Plans are to construct 10 ft of wall for every 15 ft of shoreline (i.e. 5 ft of gap between any two 10-ft wall sections).

Preloading the near shore soils prior to constructing the breakwater is also being considered to improve the soils and increase the strength of the foundation soils. Stiff clay dredged from offshore operations and placed with a hydraulic dredge will likely be used as surcharge. A year after the placement of surcharge, the imported fill will be removed and replaced with rock. Due to budget constraints, wick drains or multi stage construction for the preloading area will not be utilized.

We understand that current plans are to investigate the above-described options further by constructing 5 test sections each on the order of 500 ft in length except gravel beach and preloading sections which would be 1,200 ft and 700 ft in length, respectively.

Fugro Consultants LP (formerly Fugro South Inc.) performed a geotechnical study at the site of the proposed stabilization project to assist Shiner Moseley in their feasibility study. The results of this previous study were documented in two reports. The first report (Report No. 0602-1316, Part I of II), issued August 8, 2002, was submitted at the request of the client to aid with the conceptual designs of various shoreline stabilization structures. The second report (Report No. 0602-1316, Part II of II), issued on January 7, 2003, included settlement analyses. A total of twenty exploratory soil borings were performed for the previous study. All of the options are extremely sensitive to geotechnical parameters. This study was conducted to better define soil parameters and provide additional geotechnical recommendations to aid in the detailed investigation of the options considered for the beach stabilization project.

Scope of Work

The purposes of our geotechnical study were to: 1) explore subsurface soil conditions within the footprints of the test sections, and 2) provide geotechnical recommendations to aid in construction of the test sections and potential stabilization structures. Our scope of work included the following:

- reviewing the logs of borings presented in the report for the previous study;
- drilling and sampling nine borings to explore subsurface soil conditions across the test section area and to obtain soil samples for laboratory testing;
- performing field and laboratory tests on selected soil samples to assess pertinent geotechnical engineering properties;
- performing field vane tests at six locations to obtain insitu undrained shear strength data;
- analyzing the field and laboratory data to develop appropriate geotechnical recommendations; and
- preparing a geotechnical report summarizing our findings and recommendations.

Environmental assessment, compliance with State and Federal Regulatory requirements, assessment of potential migration, and/or environmental analyses were beyond the scope of this study. A fault study was also beyond our scope.

Applicability of Report

The scope of the explorations, tests, and analyses for this study, as well as the conclusions and recommendations presented in this report, were selected or developed on the basis of our understanding of the project, as described above and in later sections of this report. If pertinent details of the project have changed or otherwise differ from our descriptions, we request that we be notified and engaged to review the changes and, if necessary, to modify our conclusions and recommendations.

We have prepared this report exclusively for Shiner Moseley and Associates Inc. as a guide for geotechnical aspects of the design and construction of the proposed test sections for the Gulf Shoreline Stabilization Project. We have conducted this study using the standard level of care and diligence normally practiced by recognized engineering firms now performing similar services under similar circumstances. We intend for this report, including all illustrations, to be used in its entirety. This report should be made available to prospective contractors for information only and not as a warranty of subsurface conditions. Fugro makes no claim or representation concerning any activity or condition falling outside the specified purposes to which this report is directed. The observations, conclusions, and recommendations presented in this report may not apply to locations not explored by borings or areas outside the project boundaries.

Field Exploration

Our field activities are discussed in this section. We have included discussions of drilling methods, sampling methods, water depth observations, and borehole completion.

General. During the proposal phase, consideration was given to drilling borings along the centerline of the test sections in the water in an effort to obtain precise insitu shear strength data and reduce the potential for inference of soil conditions for the test sections. However, due to budget constraints, field exploration was limited to land. We explored subsurface conditions at the site by drilling 9 geotechnical soil borings to a depth of 45 feet below existing grade and performing 6 field vane tests to a depth of 20 ft each. The boring and field vane tests locations were mutually selected by Fugro and Shiner Moseley. The boring and field vane tests locations were located and staked in the field by our personnel using GPS coordinates provided by Shiner Moseley. The approximate boring and field vane test locations are shown on the Plan of Borings (Plate 2). Coordinates of field vane test locations are noted on the field vane logs. The coordinates of boring locations are tabulated below. Surface elevations for the borings or the field vane tests were not provided to us. Elevations for various proposed breakwater configurations described in this report refer to NAVD 88 datum.

BORING NO.	LATITUDE	LONGITUDE
TS-1	N 29°38’2.4”	W 92°46’23.1”
TS-2	N 29°38’4.8”	W 92°46’27.7”
TS-3	N 29°38’7.2”	W 92°46’34.4”
TS-4	N 29°38’11”	W 92°46’38.3”
TS-5	N 29°38’9.6”	W 92°46’41.2”
TS-6	N 29°38’12.5”	W 92°46’44.5”
TS-7	N 29°38’15.6”	W 92°46’48”
TS-8	N 29°38’7.2”	W 92°46’31.4”
TS-9	N 29°38’14.2”	W 92°46’46.2”

Borehole Drilling and Sampling Methods. Due to marshy surface conditions along the coast, the borings were drilled with a buggy-mounted drill rig using wet-rotary drilling techniques. Soil samples were generally taken at about 2-ft intervals for the first 16-ft, and at about 5-ft intervals thereafter to the completion depths of the borings as indicated on the boring logs. Detailed descriptions of the soils encountered in the borings drilled for this study are presented on the boring logs on Plates A-1 through A-9 in Appendix A. A key to the terms and symbols used on the boring logs is presented on Plate A-10.

Undisturbed samples of cohesive soils within the upper 16 ft were generally taken using a liner sampler. The liner samples were advanced a distance of about 24 inches using the weight of the drill string. Undisturbed samples of cohesive soils below a depth of about 16 ft were generally obtained by hydraulically pushing a 3-inch-diameter, thin-walled tube a distance of about 24 inches. Our field procedure for cohesive soil sampling was conducted in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D 1587). The samples were extruded in the field and visually classified by our geotechnical technician. We obtained field estimates of the undrained shear strength of the recovered samples using a Torvane or hand penetrometer. Where applicable, the field estimates were modified for stiff to hard, over-consolidated natural cohesive soils, as described on Plate A-10b. Portions of each recovered soil sample were placed into appropriate containers for transportation to our laboratory.

Water Depth Observations. About 5 ft of standing water was encountered at each of the boring location. Due to the low surface elevation of the coastline and the fact that wet-rotary drilling techniques were used, water depth readings within the borings could not be obtained.

Field Vane Shear Tests. Six field vane tests with a buggy-mounted remote vane system were performed to a depth of 20 ft each. The field vane tests were performed in general accordance with the *Standard Test Method for Field Vane Shear Test in Cohesive Soil* (ASTM D 2573-72). A field vane shear test is an insitu technique used to estimate the undrained shear strength of the subsurface soils. The vane was inserted into the boreholes at depth intervals of 2.5 ft. At a specific depth, the vane was rotated and the torsional force required to cause shearing was calculated. The blade was rotated at a specified rate of less than 0.1 degrees per second (practically 1 degree every 10 seconds). Results of field vane tests are presented on Plates B-1 through B-6 in Appendix B.

Borehole Completion. Each boring was sealed with cement-bentonite grout upon completion. We grouted the boreholes from the bottom up using a tremie pipe. When grout returned to the surface, we removed the tremie pipe and topped-off the boreholes by pouring grout from the surface. Our field procedure for borehole completion was in general accordance with the regulations of the Louisiana Department of Transportation and Development (LADOTD), the Office of Public Works (OPW), and the Department of Environmental Quality (DEQ).

Laboratory Testing

The laboratory-testing program for this study was directed primarily toward evaluating the classification properties and undrained shear strength of the coastal subsurface soils. We also measured the compressibility characteristics of the subsurface soils by performing five incrementally loaded consolidation tests on selected samples. Two multi-stage consolidation undrained triaxial compression tests were also performed in order to estimate soil strength gain over time. Our laboratory tests were performed in general accordance with the appropriate standards as tabulated at the end of this section.

Classification Tests. The classification tests included tests for natural moisture content, liquid and plastic limits (collectively termed Atterberg Limits), and dry unit weight. These tests aid in classifying the soils and are used to correlate the results of other tests performed on samples taken from different borings and/or different depths. The results of the classification tests are presented on the boring logs on Plates A-1 through A-9.

Undrained Shear Strength Tests. We measured the undrained shear strength of select undisturbed samples of cohesive soils by performing miniature vane shear tests. Natural moisture contents and dry unit weights were determined as routine portions of the miniature vane tests. The results of the undrained shear strength tests are presented on the boring logs on Plates A-1 through A-9.

Incremental Consolidation Tests. We measured the compressibility characteristics of the subsurface soils by performing five incrementally loaded consolidation tests. We performed each test with a rebound-reload cycle. Natural moisture contents and dry unit weights were determined as routine portions of the consolidation tests. The results of consolidation tests are presented as plots of effective vertical pressure versus strain on Plates C-1 through C-5 in Appendix C.

Multi-Stage Consolidated-Undrained Triaxial Compression Tests. Consolidated-undrained triaxial compression tests with pore pressure measurements (ASTM 4767) were performed on two soil samples as part of the program. The specimens were mounted in the triaxial cell after being extruded from the tube. Specimen saturation was achieved by back pressuring. Specimens were then isotropically consolidated. The confining stresses used were based on the estimated insitu effective vertical stresses. During loading, the necessary data (time, vertical deformation, vertical force, and pore pressure response) were recorded using an automated data-acquisition system. The specimen was loaded in a multi-stage format, allowing the specimen to approach failure under two different applied loads while collecting the data described above, then failing the sample under the third and final load increment. The results of the consolidated-undrained compression tests are presented on Plates D-1 and D-2 in Appendix D.

Summary of Laboratory Tests. The following table lists the types and number of laboratory tests as well as the standard test methods performed for this study.

Laboratory Test	Testing Standard	Quantity
Moisture Content	ASTM D 2216	70
Atterberg Limits	ASTM D 4318	27
Dry Unit Weight	ASTM D 2166	26
Miniature Vane Shear Test	ASTM D 4648	27
Multi-Stage Consolidated Undrained Triaxial Compression	ASTM D 4767	2
Incremental Consolidation Test	ASTM D 2435	5

General Site Conditions

The interpreted site and subsurface conditions based upon our field exploration, laboratory testing, and experience are discussed in this section.

Site Description. The project site is generally composed of the southern portion of Rockefeller Refuge located in Cameron Parish, Louisiana and extends westward along the beach from Joseph's Bayou approximately 10 miles to the western boundary of the refuge. Surface conditions along the coast essentially consist of very soft, highly organic topsoil and easily erodible shell fragments, which compose the beach and grassy marshland. A site vicinity map is shown on Plate 1.

Subsurface Conditions. The subsurface conditions presented in this report are based on the borings and field vane tests completed for this study and the twenty soil borings drilled for the previous study. Our interpretation of the soil stratigraphy comprises of four strata. The strata are primarily differentiated based on the soil consistency since soil strengths are critical for design. It should be noted that all nine borings drilled for this study were terminated at a depth of 45 ft. Therefore the interpretation of the soil stratigraphy for Strata III and IV is based on the soil borings drilled for the previous study.

Stratum	Soil Description	Average Depth, ft
I	Very Soft Recent Clay	0 to 20
II	Very Soft to Soft Recent Clay	20 to 40
III	Stiff Clay and Sandy Clay	40 to 60
IV	Firm to Stiff Clay and Sandy Clay	60 to at least 100

Stratum I generally consists of very soft Recent clays. The clays are highly plastic and have very high moisture contents. Measured undrained shear strengths in this stratum typically range from 20 psf to 200 psf. Undrained shear strengths as low as 20 psf were measured on several samples using torvane in the field. Average undrained shear strength in this stratum was on the order of about 85 psf. The Recent clays contained a high content of organics in the upper 16 ft. Water contents of these clays are typically very close to their Liquid Limits indicating the clays have a consistency of a thick drilling mud.

Higher undrained shear strengths in this stratum were generally measured by field vane tests as compared to those from the field torvane or the laboratory miniature vane tests. Based on our experience and a comprehensive review of data, we believe that the higher strengths are due to the skin friction mobilized along the sleeve of the field vanes. Remolded shear strengths measured from the field vanes typically ranged from 20 psf to 60 psf and were in general agreement with the remaining data.

Stratum II is generally composed of very soft to soft clay, which was encountered to a depth of approximately 40 ft below grade. We measured undrained shear strengths varying from 50 psf to 300 psf in this stratum. The highly compressible clays of this stratum have very high moisture contents and high Atterberg limits. Average undrained shear strength in this stratum was on the order of about 130 psf.

Stratum III is generally composed of stiff to very stiff clays and sandy clays. Appreciable amounts of silt and sand were encountered throughout Stratum III, which extends to a depth of approximately 60 ft below grade. Measured undrained shear strengths in this stratum generally range from 1,000 psf to 2,000 psf.

Stratum IV consists of firm to stiff sandy clays and clays, which were encountered to the termination depths of the borings drilled for the previous study. Undrained shear strengths in this stratum varied from 500 psf to 1,500 psf.

Additional information relating to the subsurface conditions encountered in the borings drilled for this project is presented on the boring logs on Plates A-1 through A-9. A key identifying the terms and symbols used on the boring logs is presented on Plate A-10.

Water Depth Conditions. As discussed previously, about 5 ft of water was encountered at each of the boring locations at the time of our exploration. Since wet-rotary drilling techniques were used for drilling, it was not possible to obtain depth-to-water readings in the boreholes. The water level is at or near the shoreline and fluctuates with currents and wave conditions in the Gulf. We used a still water level of EL +1 in our analyses.

Variations in Subsurface Conditions. Our interpretations of soil and water depth conditions, as described in this report, are based on data obtained from our visual observations, sample borings, field vane tests, laboratory tests, and our experience. Although we have allowed for minor variations in the subsurface conditions at the site, our recommendations for the Gulf Shoreline Stabilization Project may not be appropriate for subsurface conditions other than those reported herein. It is likely that some undisclosed variations in soil or water depth conditions may occur outside the boring locations. We recommend careful observations during construction to verify our interpretations. Should variations from our interpretations be found, we recommend that we be notified and authorized to evaluate what, if any, revisions should be made to our recommendations.

Soil Parameters

Different analyses including bearing capacity, slope stability, and settlement for various proposed test sections were performed for this study. Internal and global stability of the concrete panel breakwater was also evaluated. This section presents the material properties used in our analyses for insitu soils and for the proposed fill materials. The undrained and drained soil properties including the undrained shear strength, internal friction angle, effective cohesion, and effective internal angle of friction for various soil types encountered in our borings are tabulated below.

Depth Below Existing Grade, ft	Soil Description	Total Unit Weight, pcf	Undrained Condition		Drained Condition	
			Undrained Shear Strength (psf)	Friction Angle (degree)	Cohesion (psf)	Friction Angle (degree)
0 to 20	Clay	90	85	0	0	15
20 to 40	Clay	95	130	0	0	17
40 to 60	Clay and Sandy Clay	120	1,300	0	120	18
60 to 100	Clay and Sandy Clay	120	1,000	0	100	18

Material properties for proposed fill materials to be used in various test sections including total and effective unit weights, undrained shear strength and internal angle of friction for undrained conditions, and effective cohesion and effective internal angle of friction for drained conditions are tabulated on the following page.

Material	Total Unit Weight (pcf)	Effective Unit Weight (pcf)	Undrained Condition		Drained Condition	
			Undrained Shear Strength (psf)	Friction Angle (degree)	Cohesion (psf)	Friction Angle (degree)
Armor Stone	115(1)	53	N/A	40	N/A	40
Gravel	135	73	N/A	38	N/A	38
Loose Sand Fill	110(2)	48	N/A	30	N/A	30
Clay Fill	125 (above water level)(2) 115 (below water level)(2)	63 53	1000 500	N/A N/A	100 50	18 15
Light Weight Aggregate	72	9	N/A	35	N/A	35

Notes:

(1) Total unit weight of armor stone was based on discussions with Shiner Moseley and assumes a standard riprap gradation with a bulk porosity of about 35 percent.

(2) Total unit weights of sand and clay fill based on anticipated placement and compaction efforts.

Compressibility characteristics of insitu soils used in our settlement analyses were obtained from the various consolidation tests performed for this study in conjunction with the consolidation testing conducted for the previous study. These parameters including compression indices and coefficient of consolidation values are not presented in this report.

The analyses performed for this study is very sensitive to the properties of various fill materials. We selected values for use in our analyses based on our experience with similar applications. However, materials such as armor stone and gravel can vary significantly in density and can have a wide range of internal friction angles based on the source and the placement and compaction effort. Soil fill hydraulically placed under water would also vary significantly depending upon its method of placement. We strongly recommend that the fill material properties presented in this report be verified as a minimum by laboratory tests and preferably by field tests, if feasible.

Soil Bearing Capacity

The allowable bearing capacity of cohesive soils depends on the undrained shear strength of the foundation soils, and size and shape of the loaded area. We recommend an allowable bearing pressure of 220 psf be utilized for the design of shoreline protection structures placed on land that bear in the upper recent natural, clay soil deposits encountered in our exploratory borings. The allowable bearing pressure reported above includes a factor of safety of 2 with respect to shear failure of the foundation soils and is based on the proposed design configurations. However, for this application, and due to the fact that a localized failure of the protection structures on the shore may not be considered catastrophic, a lower value (1.5) may be acceptable for design. An allowable bearing pressure of 290 psf corresponds to a factor of safety of 1.5. In computation of allowable bearing capacity of the soft clays, we modeled the proposed cross-sections as strip footings due to their length to width ratios. Allowable bearing capacity of the soft clays can be increased by changing the size and shape of the breakwaters such as utilizing a square cross-section. However, we understand that although this may be possible for the test sections, actual breakwaters cannot be feasibly constructed in such a shape.

The allowable bearing pressures presented in this report are lower than the values recommended in the previous reports. This is due to the lower undrained shear strengths measured in the Recent clay deposit compared to the average values obtained for the previous study. On average, higher undrained shear strengths were measured in the soft clays, particularly in the upper 20 ft, from the field vane tests as compared to the field torvane tests and the laboratory miniature vane tests conducted for this study. Based on our experience and a comprehensive review of data, we believe that the higher strengths are due to the skin friction mobilized along the sleeve of the field vanes. Remolded shear strengths measured from the field vanes typically ranged from 20 psf to 60 psf and were in general agreement with the remaining data.

Slope and Base Stability

The overall stability of various components of the shoreline protection structures was analyzed to determine the potential of a global-type slope failure. These included base stability of various cross-sections, excavation slopes for the preloading test section and the concrete panel breakwater, and slopes of gravel and armor stone fill for the rock reef breakwater sections. We evaluated the global stability using the slope stability program, SLIDE 5.0. The program randomly generates trial failure surfaces and evaluates the factor of safety for each trial surface. The program allows a large number of potential shear surfaces to be investigated to determine the critical failure surface for each of the analyzed slope configurations. The program also allows for the analysis of slope stability using slope configurations incorporating reinforcement and load distribution techniques such as geogrids and geofabrics.

Our analyses were performed using the Modified Bishop Method for slope stability, which uses a method of slices to evaluate the stability along a series of circular failure surfaces. The computed factor of safety is the ratio of the forces resisting movement to the driving forces. A factor of safety of 1.0 or less implies the slope is unstable, while a factor of safety greater than 1.0 implies the slope is stable. An acceptable factor of safety for global stability is generally considered to be at least 1.5. Due to the very low strength of the surficial soils and the understanding that over time the onsite soils will consolidate and gain strength, we will consider a factor of safety of 1.3 to be acceptable in our computations.

Design information including land and slope geometries for various shoreline protection systems was provided to us by Shiner Moseley. The proposed design configurations used in our analyses are shown on Plates 3 through 7.

Soil Properties for Stability Analyses. The soil properties used in our slope stability analyses are tabulated in the *Soil Parameters* section of this report.

Loading Conditions. The following loading conditions were analyzed in our global stability analyses:

- Short-Term (Undrained) -- The short-term, or undrained, condition is applicable to situations before pore water pressures have dissipated, such as during and shortly following construction, as well as shortly following any significant loading. For this project, construction of shoreline protection systems may induce significant pore pressures in the insitu soils. Analyses for this condition involve the use of undrained shear strength parameters as tabulated in the preceding section.
- Long-Term (Drained) -- The long-term, or drained, case models the condition in which the pore pressures generated during construction and operation have dissipated. Analyses for this condition involve the use of drained shear strength parameters.
- Earthquake -- Considering the location of this project (in an area of very low accelerations from very infrequent, distant earthquakes), we do not believe analyses for earthquake loading is warranted.

Results of Analyses. Our analyses assumed the design configurations shown on Plates 3 through 7. Both short term (undrained) and long term (drained) loading conditions were analyzed, as described in the previous subsection, since the potential for increased pore pressures due to the construction of the shoreline protection systems could approach undrained conditions. A summary table of minimum computed safety factors for various proposed design configurations is presented below. The computer printouts from the program SLIDE 5.0 showing contours of safety factors including the most critical failure surfaces are presented on Plates 8 through 33.

Condition/Case	Undrained Condition Safety Factor	Drained Condition Safety Factor
Rock Reef Breakwater	1.68(1)	1.68(1)
Rock Reef Breakwater with LWA	1.63(1)	1.63(1)
Rock Reef Breakwater (Base Stability)	3.94	2.50
Gravel Beach Fill: Proposed Backstop Slope of 3H:1V	1.08	1.07
Gravel Beach Fill: Recommended Backstop Slope of 9H:1V	1.30	1.99
Gravel Beach Fill: Proposed Berm Slope of 5H:1V	1.18	1.32
Gravel Beach Fill: Recommended Berm Slope of 7H:1V	1.29	1.61
Concrete Panel Sand Excavation: Proposed Slope of 2H:1V	3.56	0.98(2)
Concrete Panel Sand Excavation: Recommended Slope of 5H:1V for Proposed Construction Sequence	3.74	NA
Global Stability of Concrete Panel Breakwater with Panel Embedment of El –28 ft	> 5.0	> 5.0
Preloading Test Section Excavation– Proposed Slope of 4H:1V with Berm Adjacent to Excavation	1.03	0.90
Preloading Test Section Excavation– Recommended Slope of 7H:1V with Berm 15 ft Away from Excavation	1.33	1.31
Preloading Test Section Excavation– Recommended Slope of 13H:1V with Berm Adjacent to Excavation	1.30	1.26

Notes:

1) Tabulated factors of safety correspond to shallow slope sloughing type failures since they represented critical failure surfaces for the proposed configurations.

2) Although presented, we do not consider drained conditions will govern the stability of the excavation. See text below for an explanation and recommended slope for the excavation.

Based on the computed factors of safety, we expect the rock reef breakwaters, both with the rock and the lightweight aggregate core, to be stable against a potential slope failure. The global stability of the concrete panel breakwater was also computed to be satisfactory. However, less than acceptable factors of safety were obtained for the proposed berm and backstop slopes for the gravel beach shoreline protection system. Therefore, we analyzed alternative slopes that would yield acceptable factors of safety in these cases. Slopes of 9H:1V and 7H:1V should be used for the backstop and the berm, respectively, for the gravel beach breakwater.

<u>Concrete Panel Breakwater – Excavation Slope.</u> Flattening the excavation slope for placement of sand backfill to 5H:1V may help alleviate sloughing and sediment deposition due to waves and currents, should those mechanisms be a concern. The stability of the excavation slope will be governed by undrained soil conditions since the backfill will be placed prior to the dissipation of pore water pressures from the underlying soft clays. Although, a factor of safety of greater than 1.3 was computed for a slope of 2H:1V for undrained conditions, the influence of wave action warrants careful consideration for the excavation slope and extent of actual fill. Therefore, we recommend that the excavation slope be flattened to 5H:1V. A steeper slope may be utilized if the excavation is open for a minimum period of time under controlled circumstances and is immediately backfilled with sand prior to driving the piles for the concrete panel. However, we understand that the construction sequence cannot be altered. This is because the steel sheet piles will be attached to the concrete cap and concrete panels during prefabrication thus making it infeasible to drive or jet the sheet piles through the sand wedge. A discussion on the construction sequence for the concrete panel is provided in *Concrete Panel Breakwater*.

<u>Preloading Test Section – Raised Area.</u> The stability of side slopes of the area raised from the imported fill for the preloading test section was not evaluated in our analyses. This is due to the fact that building the raised area within a short period of time to its proposed height will likely cause a bearing capacity failure in the underlying soft clays. As such, the slopes of the raised land will not be governed by the classical theory of balancing the driving and resisting forces along a circular failure surface. A bearing capacity failure of the insitu clays may occur in several modes including general shear, punching shear, and local shear. As discussed in *Preloading Test Section*, the shape and size of a bearing capacity failure wedge cannot be accurately predicted. However, the bearing capacity failure will likely be a combination of several modes described above and will result in excessive differential movements across the width and length of the raised area. Lateral displacement of underlying soft clays, i.e. a mudwave, will also possibly occur. To reduce differential movements (within the raised area) associated with a bearing capacity failure, we recommend that the side slopes of the area be built as flat as possible. A minimum slope of 5H:1V as compared to the proposed slope of 2H:1V is suggested for the raised land.

<u>Preloading Test Section – Berm Position and Excavation.</u> We analyzed the proposed excavation slope of 4H:1V for the preloading test section with the approximately 3 ft high berm (made from the excavated soils) located within a few feet of the crest of the excavation slope. A less than acceptable factor of safety was calculated for this configuration. We modeled several slope configurations, offsetting the distance of the berm from the crest of the excavation slope while varying the excavation slope to determine configurations that would provide a factor of safety of at least 1.3. Based on our analyses, we recommend either a) the excavation slope be built at 7H:1V

with a minimum distance of 15 ft between the toe of the berm and the crest of the excavation slope or b) the excavation slope be built at 13H:1V with the toe of the berm located within a few feet of the crest of the excavation slope. The berm was analyzed using side slopes of 5H:1V and a crest width of 75 ft.

Base Stability. The minimum computed factors of safety for the rock reef breakwaters were typically associated with failure surfaces along the slope faces in our analyses. Higher factors of safety for failure surfaces passing through the underlying very soft clays were obtained. These failure surfaces correspond to base stability evaluation of the protection systems. The base stability of the rock reef breakwaters is improved because of the utilization of a geofabric/geogrid composite beneath the rock and the lightweight aggregate. Critical failure surfaces for the sand backfill excavation for the concrete panel breakwater and for the berm and backstop slopes for the gravel beach breakwater extend through the underlying very soft clays. Although, we have not analyzed the factors of safety for these cases, we expect that the base stability of these configurations can be also improved by incorporating a geofabric/geogrid composite into the system. If economically feasible, we would be glad to evaluate the stability of these systems with the utilization of the geofabric/geogrid.

As discussed in *Preloading Test Section – Raised Area*, a base failure is expected beneath the raised area for the preloading test section if built to its proposed height within a short period of time. We do not expect the base stability of the preloaded area can be improved to an acceptable level by utilizing a simple geofabric/geogrid composite beneath the area.

Shallow Localized Failures. Shallow localized failures along the face of the breakwater slopes may occur over time due to sloughing and reveling in the armor stone/gravel. However, these failures can be addressed periodically and as such, we do not consider these failures to be detrimental to the overall integrity of the slopes. Therefore, we have not analyzed the slope stability for these isolated shallow failures.

Settlement of Breakwater Systems

Design configurations of various shoreline protection systems were provided to us by Shiner Moseley and are presented on Plates 3 through 7 at the end of this report. We computed the total and differential settlements for these sections based on the proposed configurations. Properties of the fill materials tabulated in *Soil Parameters* were used to calculate applied pressures beneath the proposed shoreline protection systems. Total and effective unit weights of fill materials were calculated based on a water level elevation of EL +1 on both sides of the breakwaters.

Estimation of settlement for the very soft clays encountered at this site is difficult. Collection and testing of the insitu soils proved onerous without developing sample disturbance. The settlements estimated in this section were based on available consolidation data, correlations with other engineering properties, judgment, and our past experience with similar soils.

Methods of Analyses. To evaluate settlement of very soft clays, we performed settlement analyses using our in-house computer program SETANL. This program first computes net stress changes at selected locations and depths beneath loaded areas using Boussinesq theories of stress distribution. The program then uses soil compressibility parameters for cohesive soils to evaluate the change in thickness of individual layers and compute the overall movement at selected locations. Soil compressibility parameters used in our analyses were developed using laboratory consolidation test data presented on Plates C-1 through C-5 in Appendix C. The following assumptions were made in our analyses.

- Soil stratigraphy is assumed to have an infinite extent,
- Settlement is only under the load of the fill materials placed as part of the shoreline protection systems, and
- Significant disturbance to the insitu soils will not occur during construction.

We understand that the entire area over which the breakwaters will be constructed was previously emergent land with a ground surface elevation of at least El +1 ft. Due to erosion, the mudline has been reduced to about EL -4 ft. The loss of soil has resulted in a decrease of applied load to the underlying soils in the immediate area of the proposed breakwaters. Based on this previous loading history, we reduced the total applied loads to the soils within the entire footprints of the breakwaters by a value of 140 psf to determine the net applied loads for settlement determination. For example, if the total load from the new breakwater was 280 psf, we reduced this value by 140 psf for the net applied load and used this value to determine magnitude of settlement. Applied loads were not reduced as described above beneath the gravel beach backstop due to the existing grade within the footprint of the backstop. Computed center and edge settlements for various proposed cross-sections are tabulated on the following page.

Shoreline Protection Alternatives	Crest Elevation (feet)	One-Dimensional Consolidation	
		Estimated Center Settlement Along Length of Cross Section (feet)	Estimated Edge Settlement Along Length of Cross Section (feet)
Rock Reef Breakwater	-1.0	0.7 to 0.8	0.3 to 0.4
	0.0	0.9 to 1.1	0.5 to 0.6
	+ 1.0	1.1 to 1.3	0.7 to 0.8
Rock Reef Breakwater with Lightweight Aggregate	+2.25	0.8 to 1.0	0.5 to 0.6
	+3.25	1.3 to 1.5	0.7 to 0.9
Gravel Beach Fill (Backstop)	+ 6.0	2.0 to 2.2	1.2 to 1.4
Gravel Beach Fill (Berm)	+ 2.0	1.2 to 1.4	0.6 to 0.7
Concrete Panel (Assuming Sand Placement to El -9)	-0.5	1.1 to 1.2	0.6 to 0.7
Preloading Test Section[(1)]	+10.0	4.0 to 4.5	2.5 to 2.9

Notes:

(1) Settlements are only based on consolidation one-dimensional analyses and do not take into account lateral and differential movements due to an expected bearing capacity failure.

Time Rate of Settlement. The settlements tabulated above are long-term consolidation settlements and will occur over a period of <u>decades</u> as the highly plastic, very soft soils consolidate. In general, the consolidation process begins with the application of load on the soil resulting in excess pore water pressures. The pore water pressures gradually dissipate due to the expulsion of water from the soil voids accompanied by a concurrent increase in the effective stress of soils. As the pore water dissipates, the load is transferred to soil particles compressing the soils and resulting in settlement. The rate of settlement is a function of soil characteristics such as compressibility, permeability, and the distance of the drainage path for the excess pore water to travel. For this site, due to the lack of granular strata or sand and silt seams in the subsurface soils, pore water pressures generated during loading will dissipate very slowly, therefore, greatly reducing the rate of settlement. Based on computed coefficient of consolidation values from the consolidation test results, we expect about 40 to 50 percent of total settlement will occur over a period of about 8 to 12 years. The remaining settlement will likely occur over a period of 40 to 45 years. Settlements could be greatly accelerated by providing a drainage mechanism within the very soft clays. Wick drains are typically used for this purpose. A discussion on wick drains is provided in *Preloading Test Section*.

Settlement Monitoring. Settlement monitoring can be used to compare predicted and actual settlements, and may be used to estimate future volumes of fill needed to achieve proper site grade. We recommend that the results of the settlement monitoring program be reviewed by the Geotechnical Engineer during the construction phase. Since the instrumentation to be used in a settlement monitoring program will require experienced personnel to install, we recommend that the proposed installer have at least 5 years experience on similar projects, and that the work be performed under the supervision of the Geotechnical Engineer.

Instrumentation for the monitoring program should include the following: settlement plates, Sondex settlement systems, and pneumatic piezometers. The settlement instruments should be placed at critical locations such as below the center and edges of the breakwaters.

Settlement plates are used to monitor near-surface settlements whereas Sondex tubes can be used to monitor settlements at various depths within the upper 20 to 40 ft of normally consolidated clays. Pneumatic piezometers can be used to obtain the rate of dissipation of pore pressures with time, which will in turn be used to evaluate the magnitude of settlement as well as time rate of settlement. The proposed instrumentation must be installed prior to placement of fill for preloading section to provide meaningful data. As such, the instrumentation must be modified to accommodate the grade increase as t occurs and must also be protected from construction equipment during fill placement. We recommend that the monitoring of the instrumentation and subsequent evaluation be performed by a geotechnical engineer.

Shoreline Protection Alternatives

The following sections provide our geotechnical related recommendations for the proposed shoreline protection systems. These alternatives include a) rock reef breakwater with gravel beach, b) rock reef breakwater with lightweight aggregate, c) concrete panel breakwater, and d) preloading section. To simplify and to minimize repetition, the rock reef breakwaters are discussed under a single section titled *Rock Reef Breakwaters*.

<u>Rock Reef Breakwaters</u>

Two proposed shoreline protection systems will incorporate rock reef breakwaters. Rock reef breakwaters are low and broad crested structures, which are designed to decrease the wave energy impacting the shoreline, but still allow some transmission under day-to-day conditions. The two alternatives include a rock reef breakwater with a gravel beach and a rock reef breakwater with lightweight aggregate. The first alternative will consist of a rock reef breakwater located in front of a gravel beach. Armor stone placed over a geofabric/geogrid composite over the soft clays will form the rock reef breakwater. The base width of the cross-section will be about 64 ft whereas the crest width will be about 30 ft. Several crest elevations varying between El –1 and El +1 are currently being considered to assess their impact on the performance of the breakwater. The gravel beach will have two portions, a backstop and a berm gradually extending into the water. The approximately 3 to 4 ft thick backstop will have a crest width of 30 ft with proposed side slopes of 3H:1V. The gravel berm will have a crest elevation of El +2 and a proposed side slope of 5H:1V. The second alternative, rock reef breakwater with lightweight aggregate, will be essentially similar to the rock reef breakwater with the exception that it would contain an inner lightweight aggregate core. The lightweight aggregate core will reduce the applied bearing pressure on the underlying soft clays and thus allow for a higher crest elevation for overlying armor stone. Crest elevations of El +2.25 and EL +3.25 will be considered in design. Cross-sections of the proposed design configurations for the various rock reef breakwater options are shown on Plates 3 through 5.

Settlement of Rock Reef Breakwaters. The rock reef breakwaters will settle as the underlying soft clays consolidate over a period of decades. Estimated settlements for the two rock reef breakwater alternatives are presented in *Settlement of Breakwater Systems*.

Stability of Rock Reef Breakwaters. The stability of rock reef breakwaters, both base and slope stability, will be critical in determining design configurations for the breakwaters. We analyzed stability of proposed configurations to determine factors of safety against failure. For cases where less than acceptable factors of safety were calculated, we recommended configurations that would yield acceptable factors of safety. A detailed discussion on the stability of the rock reef breakwater is provided in *Slope and Base Stability*.

We recommend that the sequence of the breakwater construction be such that the entire breakwater is constructed in relatively uniform lifts. Significant (more that about 1 ft) differences in height during construction should be avoided to reduce the potential for slope/base failures.

Geofabric/Geogrid Composite. Plans are to incorporate a geofabric/geogrid composite into the rock reef breakwaters. The composite would need to provide two functions; to act as a separator between the heavy rock and the very soft clays and to increase the load carrying capabilities of the very soft clays by distributing the weight of the fill soils over a larger area than if no geotextiles were used. The geofabric should be placed in good contact with the mudline and should allow passage of water while retaining the overlying rock. The geofabric should be inert to commonly encountered components of salt water. A bi-axial geogrid should be placed on top of the geofabric. We expect that a geofabric such as Mirafi 600X or equivalent along with a geogrid such as Tensar BX 1100 will provide the required functions. Other composite geotextiles similar to those described above may also be available. We recommend that the geotextile manufacturer be consulted for the selection of the proper geotextiles to be used for this particular application. The geotextile manufacturer can also provide guidelines for placement techniques and required overlapping of materials.

Containment of Lightweight Aggregate Core. The rock reef breakwater with lightweight aggregate core will require containment for the almost buoyant aggregate and to reduce losses due to wave action and construction activities. Geotubes are relatively large diameter cylindrical synthetic tubes that can be filled with dredge materials. Geobags similar to geotubes may also be available. The geotextile manufacturer should be consulted for the selection of the proper geofabric to be used for this particular application. Similar breakwaters in Louisiana have been successfully completed using shell or lightweight aggregate as the core. Care must be exercised during placement to protect the lightweight aggregate from wave action both during placement and after construction. Typically a geofabric/geogrid composite is placed by hand on the mudline, lightweight aggregate is deposited by a barge and dredge lines, a new geofabric is placed atop the lightweight aggregate and armor stone is placed to a determined thickness atop the upper fabric. We expect that a line of barges or similar wave break could be utilized for protection during placement.

Lateral Soil Displacement. We have been requested to provide an estimate of immediate soil settlements upon construction of breakwaters in order to estimate the volume of materials required for maintenance. As discussed before, consolidation settlements of the breakwaters will occur at a very slow rate. About 10 percent of the total settlement may occur within 4 to 6 months of construction. However, lateral soil displacement (mudwave) would likely be created in the very soft clays due to impact from rock placement. The magnitude of displacement will depend upon the drop height of the rock, localized bearing capacity of the soil, and the size of the rock. We are not aware of any methods to compute the magnitude of such displacements. However, based on our engineering judgment, we expect that lateral displacements on the order of several feet (wave length of mudwave) may occur immediately upon placement of rock. Due to lateral squeezing of soft clays, the rock may sink a few inches into the underlying clays. We recommend that the height from which the materials are dropped into the water be controlled to reduce the extent of lateral displacement. It would be prudent to gently place the rock on the subgrade, subsequent to the installation of geofabric and geogrid, as opposed to dropping the rock.

Concrete Panel Breakwater

The concrete panel breakwater design will consist of 40 ft long sections of precast concrete panels with interbedded steel sheet piles. Each of these sections will be supported on two square concrete piles driven into the stiff clays. A portion of the soft clays on either side of the concrete panel will be replaced with imported sand to reduce the active earth pressures on the panel and provide increased passive resistance in front of the panel. The sand will be capped with armor stone to complete the breakwater. A cross-section of the proposed design is shown on Plate 6.

Sequence of Installation. An excavation will be initially performed in the very soft clays to El -9 ft to a lateral extent sufficient of satisfying the active and passive wedges of the soil. The 16-inch square precast concrete piles will then be driven into the underlying stiff clays to provide axial support for the concrete panel. The panels will be subsequently set on top of these concrete piles. The steel sheet piles will likely cut into the soft clays without the need for driving. Once the cap is set, the sand will be backfilled into the open excavation and capped with protective armor stone.

As discussed in *Concrete Panel Breakwater – Excavation Slope* under *Slope and Base Stability* for the proposed construction sequence, if sloughing due to wave action is a concern, a flatter excavation slope of 5H:1V may be employed thus requiring a higher volume of sand. The internal stability of the concrete panel will also be temporarily reduced until the excavation is backfilled. A steeper slope may be utilized if the excavation is immediately backfilled with sand prior to driving the piles and setting the concrete panels. This change in sequence would require less volume of sand backfill. However, we understand that the construction sequence cannot be altered. This is because the steel sheet piles will be attached to the concrete cap and concrete panels during prefabrication thus making it infeasible to drive or jet the sheet piles through the sand wedge.

Pile Capacity. We recommend the concrete piles be driven to a minimum depth of El -50 ft into the stiff clays. However, since the piles will need to carry the weight of the concrete panel and the steel sheet piles in addition to their own weight, a deeper depth of embedment will likely be required. For concrete piles designed with a factor of safety of at least 2.0, we expect the piles will settle less than about ½ to 1 inch.

<u>Negative Skin Friction.</u> Very soft clays to a depth of EL –9 ft will be excavated and replaced with hydraulically placed sand on both sides of the concrete panel to provide stability for the system. Since the weight of the backfill will exceed the weight of the removed clays, settlements beneath the sands will occur as discussed in *Settlement of Breakwater Systems*. The rate of settlement will be extremely slow and will result in additional loads being applied to the piles. The load will occur as frictional downdrag (negative skin friction) around individual piles as the soils consolidate.

Numerous techniques have been presented to evaluate the magnitude of negative skin friction. For this report, we have assumed that the theory provided by Terzaghi and Peck (1967) is applicable. This theory assumes that the maximum value of negative skin friction, which can be applied to the pile, is limited to the maximum shear force in the soil at the specified depth. To account for negative skin friction, we reduced the ultimate axial capacity presented on Plate 34 by a downdrag value of 20 kips computed for the 16-inch square piles.

<u>Static Axial Capacity.</u> The ultimate axial capacity, in both compression and tension, of a 16-inch square concrete pile was computed using the static method of analysis. In this method, the ultimate compressive capacity of a pile is taken as the sum of the skin friction on the pile and the end bearing on the pile tip. The weight of the pile is neglected in the computations. When computing ultimate tensile capacity, the end-bearing component is also neglected. To counter the effects of negative skin friction, we reduced the axial capacity by the value of downdrag noted previously.

The **ultimate** axial concrete pile capacity curve computed using the API RP 2A (1993) method[1] is presented on Plate 34. We recommend a factor of safety of 2.0 be applied to the ultimate axial capacity of concrete piles loaded in compression (transient and sustained) and transient tension. A factor of safety of 3.0 should be applied for sustained tension loads.

The uplift capacity of the piles may be increased by adding the weight of the pile. The buoyant weight of the pile should be used. A buoyant unit weight of 90 pcf is typically used for concrete. A factor of safety of 1.2 should be applied to the pile weight.

Lateral Extent of Sand Backfill. We understand that sand fill will be used to replace very soft clays on both sides of the concrete panel test section. The sand fill will lower the active pressures on the sheet pile (seaward side) due to its high internal friction angle as compared to clays while significantly increasing the passive resistance in front of the sheet pile (land side). The results of our internal stability analyses for the concrete panel indicate that about 4 times more passive resistance for the panel is provided by the sand backfill as compared to the underlying soft clays thus emphasizing the importance of sand in this application. The sand backfill would, therefore, have to extend a sufficient lateral distance (to the extent of the active and passive wedges) on either side of the sheet pile in order to rely on its intended impact on the concrete panel. The active wedge is represented by a line extending, at an angle of 45 degrees minus the effective internal friction angle of soft clay i.e. 17 degrees, from the tip of the concrete panel to the surface of the armor stone. The passive soil wedge will be represented by a line extending, at an angle of 45 degrees plus the effective internal friction angle of soft clay i.e. 17 degrees, from the tip of the concrete panel to the surface of the armor stone.

Determination of active and passive wedges of soil will depend upon the penetration of the concrete panel. Based on the proposed embedment depth of El -28 ft of the panel, the sand fill would have to extend the minimum distances shown on Plate 35 for the proposed sequence of construction. The lateral extents of sand as shown on Plate 35 are such that the hypotenuses of the active and passive wedges intersect the excavation slopes. These wedges intersect the slopes at a distance equal to one-third the length of the slope when measured from the toe of the slope. The lateral extent of sand backfill should be recomputed once a final sheet pile embedment is selected for the concrete panel. The sand should be fairly clean containing no more than about 30 percent fines.

We understand that the volume of sand required to replace the soft clays as shown on Plate 35 will likely make the concrete panel breakwater option infeasible due to its excessive cost. Reducing the volume of sand to that currently planned can be accommodated in the construction budget. However, based on the results of our internal stability analyses and those performed by Shiner Moseley using LPILE, the sand plays a critical role in providing the passive resistance to the panel.

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(1) American Petroleum Institute (1993), Recommended Practice for Planning, Designing, and Construction Fixed Offshore Platforms, API RP 2A, 20th Edition.

As discussed previously, about 4 times more passive resistance as compared to the underlying soft clays is acquired from the sand backfill. Therefore, not providing sufficient sand can significantly reduce the passive soil resistance in front of the panel. Our recommended lateral extent of sand fill is based on a worst-case scenario represented by a passive failure wedge extending from the base of the concrete panel. If true fixity, i.e. rigidity with depth, can be achieved within the concrete panel at a depth shallower than the total depth of the panel, the lateral extent of required sand can be reduced due to changed mechanics of failure. To verify the sand resistance obtained from a lesser volume of sand than recommended herein, we recommend that a full-scale lateral pile load test be performed in the field. We will be pleased to provide specifications for such a test and also review the results, if needed.

Settlement of Sand Backfill. The sand backfill and the armor stone placed on either side of the concrete panel will weigh more than the excavated soft clays. The additional weight of the sand and the overlying armor stone will cause the underlying soft clays to consolidate slowly over a period of time. Settlement estimates for the sand backfill are presented in *Settlement of Breakwater Systems*.

Stability of Concrete Panel. We were requested to evaluate the stability of the concrete panel breakwater system. The concrete panel can either be modeled as a flexible sheet pile or as a laterally loaded pile. We used the classical technique of designing cantilevered sheet piles which checks the stability of the sheet pile by balancing earth pressure forces on its both sides. Details and results of our analyses are discussed in the following sections. Shiner Moseley analyzed the concrete panel as a laterally loaded pile using the computer program LPILE Plus 4.0 developed by Ensoft, Inc. The LPILE program uses finite difference numerical techniques to compute lateral deflections and bending moments induced in a pile due to lateral and axial loads applied at the top of the pile. The pile-soil system is modeled as a series of finite segments that represent the pile and the soil. Soil resistance is provided using p-y curves developed from a distribution of input soil unit weights and strength parameters specific to the subsurface conditions encountered at the site.

At Shiner Moseley's request, our recommended parameters for use in LPILE are tabulated below.

Layer No.	Bottom Depth (ft)	Soil Type	Shear Strength (psf)	f (degree)	k (lb/in^3)	Effective Unit Weight (pcf)	e_{50}
1	2	Armor Stone	0	40	--	53	--
2	9	Medium Sand (API)	0	30	60	48	--
3	20	Very Soft Clay	85	0	--	28	0.030
4	40	Very Soft Clay	130	0	--	33	0.030

Internal Stability Analyses–Classical Approach. Although, the proposed concrete panel breakwater will be a complex system with different elements such as piles, concrete panels, and steel sheet piles of varying depths, we modeled the system as a cantilever bulkhead in order to simplify it. The driven concrete piles due to their large inter pile spacing were neglected in our stability analyses. In general, two types of failure mechanisms are considered in designing bulkheads: 1) internal bending or toe-kickout of the sheeting (local failure) and 2) overall bulkhead and foundation soils failure (global failure). A local failure is usually a result of inadequate design of the sheet pile (penetration or section modulus) to retain the soil behind it. Generally, a failure in this category results in large deflections at the top in cantilevered bulkheads. Global stability depends on the topography in the area of the bulkhead in addition to the soil properties, and generally results in a rotational type failure of the entire bulkhead system and the subsurface soils.

In our design procedure, we first typically design the bulkhead to resist local failure i.e. sheet pile penetration and required section modulus. After these parameters are defined, we evaluate the selected cross-section for global stability. In cases, where global stability is critical we increase the length of the sheets until an acceptable factor of safety is obtained.

Internal Stability-Method of Analysis. We analyzed the concrete panel using the Corps of Engineers' computer program CWALSHT[(2)]. The program CWALSHT uses classical methods for the design and analysis of anchored or cantilevered sheet pile walls. We performed our analyses using both short-term (undrained) and long-term (drained) soil strength parameters. The water level was assumed to be at EL +1 on both sides of the concrete panel in our analyses. At the request of Shiner Moseley, we performed a sensitivity analyses for the concrete panel for a range of wave loads to determine its impact on the panel requirements. For the highest wave load of 7.5 Kips/ft of wall, we also analyzed the concrete panel for two different thicknesses of sand backfill to assess its influence on the panel requirements. We did not analyze the internal stability of the concrete panel for the construction case as it is expected to take place in controlled circumstances with minimum wave action acting on the panel.

We determined the maximum bending moments by analyzing the sheet pile wall for a factor of safety of 1.0 for both the active and passive pressures. We then determined the required penetration by analyzing the sheet pile wall using a factor of safety of 1.0 for active pressure and a factor of safety of 1.5 and 1.25 for passive pressures. We used the Free Earth Method for cantilevered sheet pile walls to determine maximum bending moment and required penetration.

Results of CWALSHT Analyses. Detailed results of our analyses for the internal bulkhead stability for both undrained and drained conditions are tabulated on the following page.

(2) Department of the Army, Waterways Experiment Station, Corps of Engineers. (1991), *"Computer Program for Design and Analysis of Sheet-Pile Walls by Classical Methods (CWALSHT) Including Rowe's Moment Reduction,"* Vicksburg, MS.

Case	Loading Condition	Bottom EL. of Sand Fill (ft)	Wave Force (lb/ft)	Safety Factor		Bottom EL. of Wall (ft)	Maximum Bending Moment (ft-lb)
				Active Pressure	Passive Pressure		
No Sand Fill	Undrained	N/A	7,500	1.0	1.5	-63	N/A
				1.0	1.25	-59	N/A
				1.0	1.0	N/A	87,475
	Drained			1.0	1.5	-42	N/A
				1.0	1.25	-40	N/A
				1.0	1.0	N/A	72,143
Sand Fill	Undrained	-9	7,500	1.0	1.5	-59	N/A
				1.0	1.25	-54	N/A
				1.0	1.0	N/A	66,064
	Drained			1.0	1.5	-37	N/A
				1.0	1.25	-34	N/A
				1.0	1.0	N/A	54,210
Sand Fill	Undrained	-15	7,500	1.0	1.5	-49	N/A
				1.0	1.25	-46	N/A
				1.0	1.0	N/A	66,064
	Drained			1.0	1.5	-31	N/A
				1.0	1.25	-29	N/A
				1.0	1.0	N/A	53,522
Sand Fill	Undrained	-9	5,000	1.0	1.5	-42	N/A
				1.0	1.25	-35	N/A
				1.0	1.0	N/A	37,019
	Drained			1.0	1.5	-29	N/A
				1.0	1.25	-26	N/A
				1.0	1.0	N/A	30,868

Case	Loading Condition	Bottom EL. of Sand Fill (ft)	Wave Force (lb/ft)	Safety Factor		Bottom EL. of Wall (ft)	Maximum Bending Moment (ft-lb)
				Active Pressure	Passive Pressure		
Sand Fill	Undrained	-9	5,300	1.0	1.5	-45	N/A
				1.0	1.25	-41	N/A
				1.0	1.0	N/A	40,168
	Drained			1.0	1.5	-29	N/A
				1.0	1.25	-27	N/A
				1.0	1.0	N/A	33,384
Sand Fill	Undrained	-9	6,000	1.0	1.5	-49	N/A
				1.0	1.25	-45	N/A
				1.0	1.0	N/A	47,870
	Drained			1.0	1.5	-32	N/A
				1.0	1.25	-30	N/A
				1.0	1.0	N/A	39,486
Sand Fill	Undrained	-9	7,000	1.0	1.5	-56	N/A
				1.0	1.25	-51	N/A
				1.0	1.0	N/A	59,720
	Drained			1.0	1.5	-36	N/A
				1.0	1.25	-32	N/A
				1.0	1.0	N/A	48,959

Note: The required penetration of the concrete panel should be based on both internal and global stability analyses and should have an adequate factor of safety against both local and global type failures.

<u>Global Stability of Concrete Panel.</u> The global stability of the concrete panel was analyzed to determine the potential of a global type failure. Soil parameters used in our analyses, method of analyses, and the results of analyses are discussed in *Slope and Base Stability*. We analyzed the stability of the proposed concrete panel with an embedment depth of El –28 ft. Increasing the embedment of the concrete panel will increase the global stability of the panel. Since acceptable factors of safety were obtained for an embedment depth of El –28 ft, no further global stability analyses were performed for alternative depths of sheet pile. In our analyses, we assumed failure surfaces could not intersect the concrete panel sheets. We also assumed the concrete panel to be continuous along its length and did not account for the shorter steel sheet piles linking the panels. Furthermore, the driven concrete piles intended to support the concrete panel were ignored in our analyses because of their large spacing. The water level was assumed to be at EL +5 on the seaward side and at EL +1 on the land side in our analyses. Shallow failures extending beneath the shorter steel sheet piles may occur over time as the soft clays squeeze from one side of the

panel to the other. However, these failures can be addressed periodically and as such, we do not consider them to be detrimental to the overall stability of the concrete panel.

Concrete Panel Breakwater Design. As described earlier, the concrete panel should be designed with an adequate factor of safety against both local failure and global failure. The concrete panel penetration and section requirements will be controlled by the internal stability rather than the global stability for the proposed configuration. Typically, a minimum factor of safety of 1.5 with regards to the internal stability of the sheet pile is required to select the sheet pile embedment depth. We recommend that the concrete panel penetration be selected based on a factor of safety of 1.5 for a wave load representative of typical storm conditions. A factor of safety of 1.25 may be used to select the concrete panel penetration for the maximum design wave load of 5.3 kips/ft of wall, representative of an approximate 100-year storm event. The type of section required to resist the bending moments in the sheet pile should be based on the maximum bending moment computed for either of the undrained or drained conditions and corresponding to the appropriate case used for the selection of the sheet pile penetration. For example, a sheet pile embedment of El -41 ft and a maximum bending moment of about 40 Kip-ft should be used for the case with sand backfill to EL -9 and a wave load of 5.3 Kips/ft of wall.

Preloading Test Section

The preloading test section will consist of an approximately 100 ft wide by 700 ft long area, which will be preloaded with the intent of surcharging the soft clays and improving the strength of the subsurface soils. This section will be built on land. An excavation will be initially performed to El -4 ft to remove the surficial soft clays. Excavated materials will be placed adjacent to the crest of the excavation slopes on all four sides of the excavation in the form of berms. The approximately 3 ft high berms will be 75 ft long and will have side slopes of 5H:1V. The excavation will be backfilled with stiff clays to be brought in from offshore dredging operations. The stiff clays will be hydraulically placed from a barge. Finished crest elevation of the raised area will be about El +10 ft thus incorporating about 14 ft of imported fill material. Plans are to remove the surcharge fill from the raised area, one year after construction of the test section, and replace it with a thickness of armor stone that would yield an equivalent weight to that of the removed fill. This section would then serve as a breakwater in a similar capacity to the rock reef breakwater. A cross-section of the proposed design is shown on Plate 7.

Stability of Preloaded Area. The intended purpose of preloading the onsite soils is to accelerate settlements in the very soft clays. In order for this test section to work, the surcharge soils would have to cause majority of total settlements to occur within a short period of time. Subsequent to replacing the imported fill with rock (one year after the construction of the breakwater), little maintenance would be required over the life of the structure as the post-construction consolidation settlements are reduced.

Proper Technique of Preloading. The proper technique to build the preloaded area should include placing fill to a height that would apply a bearing pressure on the underlying soils equal to or less than their ultimate bearing capacity. The soils should then be allowed to consolidate and gain strength as the pore water pressures are expelled from the soil voids over time. As the soils gain strength, their bearing capacity increases. Further fill should then be placed to a height controlled by the increased soil bearing capacity. For this case, it would require that the fill be placed in several stages. As discussed in *Time Rate of Settlement* under *Settlement of Breakwater Systems*, actual settlements are expected to occur very slowly due to the high plasticity and moisture contents of the insitu clays. We expect that only 10 percent of total settlements will occur within 4 to 6 months of construction with the remaining settlements occurring over a period of decades. As such, without utilizing external mechanisms to increase the rate of settlement, soils would require a long time to gain strength thus significantly delaying the time period between stages of fill placement. Based on this reasoning, wick drains will be required to accelerate settlements. A discussion on wick drains in presented in *Wick Drains*.

We understand that the construction budget for the test section does not allow for multiple contractor mobilizations that would be required for fill placement in stages. Furthermore, wick drains due to their associated cost cannot be incorporated into the test section. We have been requested to comment on the consequences of building the test section as proposed. A discussion on base and slope stability, settlement, and lateral displacement for the proposed test section is provided in the following subsections.

<u>Wick Drains.</u> Wick drains are typically used with a surcharge applied at the ground surface. Vertical wick drains typically consist of a fabric wrapped plastic wick that is installed into the soft, weak soils. To facilitate consolidation of the soft soils, a surcharge load is applied across the site after the installation of wick drains. The required height of the surcharge depends on the final design criteria and expected post construction performance. The length of time required for surcharging can be determined once final design criteria has been selected. Normally, surcharging in conjunction with wick drains can range from 1 year to more than 3 years depending on the design criteria and the actual performance of the wick drains in the field.

The water collected by the wick drains generally travels to the surface or other drainage layer and is typically retrieved in a collection system. The collection system usually includes a sump and a pump and a drainage path to remove water transmitted through wick drains. Wick drains installed at this site would likely need to be placed at 3 to 5 ft centers. The wick drain manufacturer should design the final type, depth, and spacing of wick drains to achieve the desired preconsolidation of the soft clays.

We estimate that it would take about 1 to 2 years for a significant portion of the consolidation to occur. Additional consolidation settlements would continue to occur long after the primary consolidation is complete. After improving the subgrade soils through the use of wick drains, the surcharge load could be removed and replaced with armor stone.

Base Stability. We computed an applied pressure of greater than about 1,200 psf within the footprint of the surcharged area. The **ultimate** bearing capacity of onsite soft clays was computed to be 440 psf as discussed in *Soil Bearing Capacity*. The applied pressure exceeds the **ultimate** bearing capacity by a factor of almost 3 and the allowable bearing capacity by a factor of almost 4.5 even for the low acceptable factor of safety of 1.5. As such, surcharging the area as planned will likely cause a bearing capacity failure in the insitu soft clays. A bearing capacity failure may occur in several modes including general shear, punching shear, and local shear. In general, the failure mode depends on the relative compressibility of the supporting soils. The three types of failures are summarized below.

- General Shear – General shear is characterized by the formation of a well-defined failure pattern with considerable bulging of the soil mass near the surface. The slip surface starts emerging on the edge of the loaded area towards the ground surface. In clays, the surface of sliding generally terminates at the boundary of the zone of elastic equilibrium. In some soils, the failure of the loaded area may be sudden and is usually accompanied by substantial tilting of the area.

- Punching Shear – In this mode of failure, the vertical movement of the loaded area is accompanied by a compression of the soil immediately underneath the loaded area. The loaded area continues to penetrate by vertical shear around its perimeter and is characterized by sudden movements. The soil outside the loaded area remains relatively uninvolved and movements on the side of the area are relatively small. Visible collapse and substantial tilting are rarely observed.

- Local Shear – In this mode, a wedge and slip surfaces develop starting from the edges of the loaded area. This type of failure is associated with rapidly increasing settlement resulting in significant vertical compression. The slip surfaces typically end within the soil mass until the vertical displacement of the loaded area attains a value of about one-half the width of the loaded area before the slip surfaces propagate to the ground surface. Thus, the local shear has some characteristics of both the general and punching modes of failure. For a large loaded area, localized edge bearing failure may result when soft soils are present beneath the edge of the loaded area.

We expect that a bearing capacity failure for the test section will likely be a combination of all three modes discussed above. We have been requested to provide an approximate delineation of the failure surface. We are not aware of any method, which can predict the shape and size of a bearing capacity failure. A punching shear failure may represent the best-case scenario where the loaded area punches through the soft clays causing little effect on surrounding soils and producing relatively uniform movements. A more likely scenario would, however, entail excessive differential movements across the width and length of the loaded area along with soil bulging near the surface. Failure of the soft clays would also cause the soils to be remolded thus further decreasing the bearing capacity of these soils. Remolded strength of soft clays could be as low as one half of the peak strength. As such, removal and replacement of imported fill with an equivalent weight of rock (one year after construction) as planned may cause additional bearing capacity failures although no additional weight would be added. Due to these factors, we do not believe that a rock breakwater can be successfully constructed utilizing the preloading section as proposed.

Note that the **ultimate** bearing capacity presented above is based on a test section, 100 ft wide by 700 ft long in plan dimensions. We understand that the length of the test section may be revised to 200 ft. A higher bearing capacity will be obtained for a shorter test section. However, we do not recommend building a test section shorter than about 500 ft as it would not simulate a real breakwater. A shorter test section will undergo less settlements, will have higher bearing capacity, and will likely cause less lateral displacement effects as compared to a real breakwater thus producing misleading performance results.

<u>Slope Stability.</u> We evaluated the stability of the excavation slope for the preloading test section. Results of our analyses along with recommendations regarding the slope and the location of the berm (made from the excavated soils) are discussed in *Preloading Test Section – Berm Position and Excavation* under *Slope and Base Stability*. A discussion on the side slopes of the preloaded area is presented in *Preloading Test Section – Raised Area* under *Slope and Base Stability*.

Settlement of Preloaded Area. Estimated settlements of preloaded area are presented in *Settlement of Breakwater Systems*. Note that estimated settlements are only based on one-dimensional consolidation and do not take into account lateral and differential movements across the test section due to an expected bearing capacity failure of the underlying soft clays.

Lateral Soil Displacement. Lateral displacement of soft clays i.e. a mudwave will possibly occur during the placement of fill soils due to impact loading from the soil. These movements will be independent of consolidation settlements, which would occur over a long period of time. The magnitude of displacement will depend upon the drop height of the material and localized bearing capacity of the soil. We are not aware of any methods to compute the magnitude of such displacements. However, based on our engineering judgment, we expect that lateral displacements on the order of several feet (wave length of mudwave) may occur immediately upon placement of soils. Due to lateral squeezing of soft clays, the imported fill may sink a few inches into the underlying clays. We recommend that the height from which the materials are dropped into the water be controlled to reduce the extent of lateral displacement. We do not expect that the lateral displacements generated from the preloading area will affect the closest test section, i.e. concrete panel breakwater. The concrete panel breakwater will be located about 300 ft from the preloading test section.

Construction Equipment. We expect that construction for the rock reef breakwater and the concrete panel test section will be performed by equipment placed on barges. Construction for the preloading test section will, however, include construction equipment on the beach. This equipment should be carefully selected and should impart very low bearing pressure on the subgrade soils. Remolding of the soils and continued operation of the construction equipment may further reduce the bearing capacity of the soils. Construction equipment may sink in the very soft clays at this site unless it is supported by mats or other properly prepared subgrade.

Construction Monitoring

We recommend that a geotechnical engineer, or qualified representative, be present on-site to observe the construction of shoreline protection structures. On-site observations may aid in recognizing and reconciling any unanticipated soil or groundwater condition and to check that design recommendations are appropriate and properly implemented during construction. During the construction phases, we can provide construction surveillance to: (1) observe compliance with the design concepts, specifications, and recommendations; and (2) observe subsurface conditions during construction.

* * *

The following illustrations and appendices are attached and complete this report:

ILLUSTRATIONS

APPENDIX A

APPENDIX B

APPENDIX C

APPENDIX D

Closing

We appreciate the opportunity to be of continued service to Shiner Moseley and Associates and look forward to working with you again in the near future. Please call us if you have any questions or comments concerning this report or when we may be of further assistance.

Sincerely,
FUGRO SOUTH, INC.

Jun Wang, Ph.D.
Graduate Engineer

Mohammad T. Haider, P.E.
Project Manager

Copies Submitted: Addressee (3)

JW/MTH
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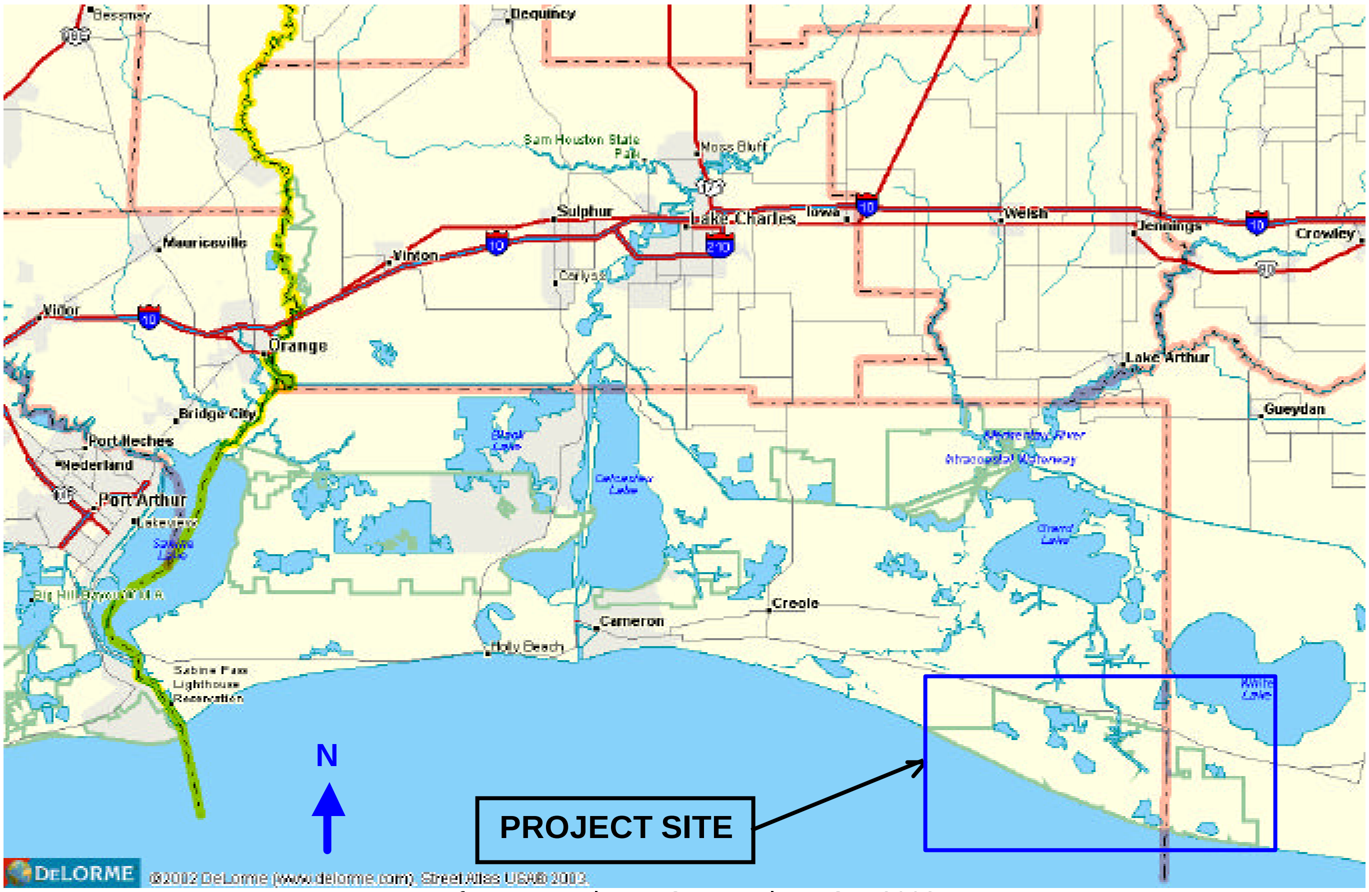
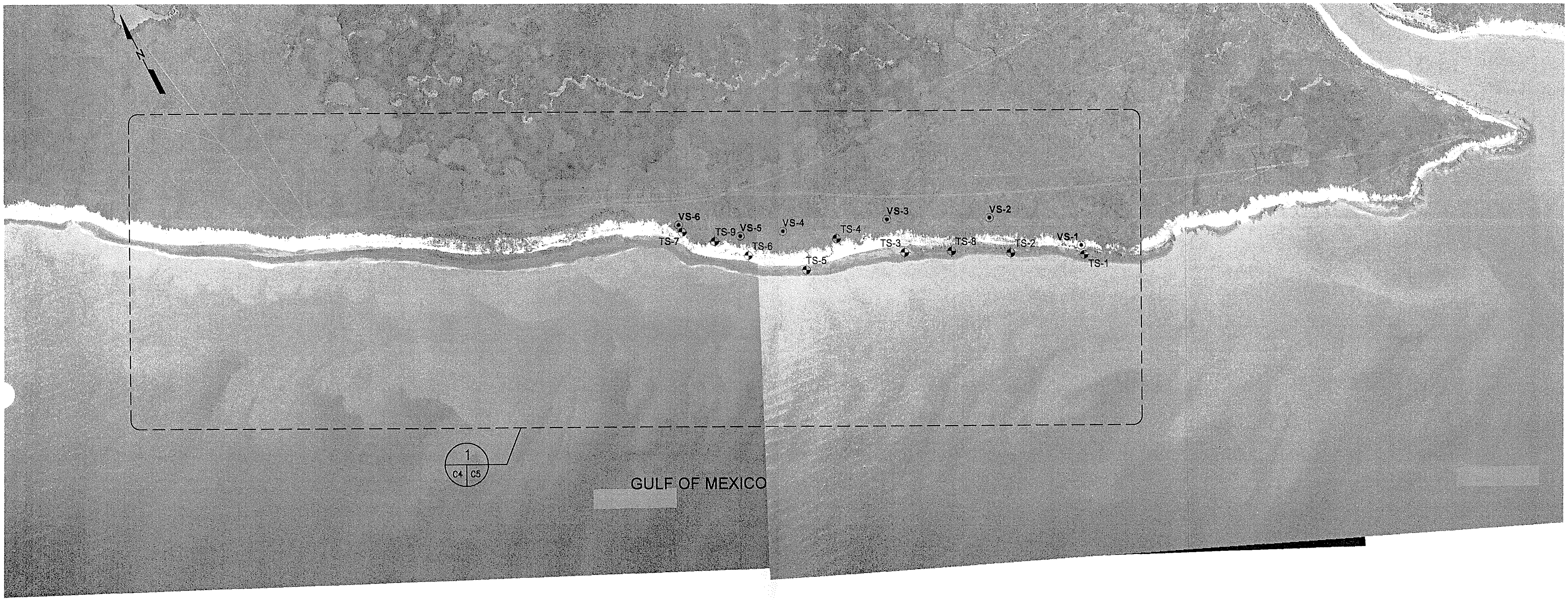
ILLUSTRATIONS



Reference: Delorme Street Atlas USA, 2003

VICINITY MAP
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA
(NOT TO SCALE)



LEGEND

TS-5 APPROXIMATE LOCATION OF SOIL BORING

VS-6 APPROXIMATE LOCATION OF FIELD SHEAR VANE TESTS

NOTE

AERIAL PHOTOGRAPH SHOWN ON THIS SHEET WAS TAKEN JULY 18, 2002 BY LANMON AERIAL PHOTOGRAPHY, INC.

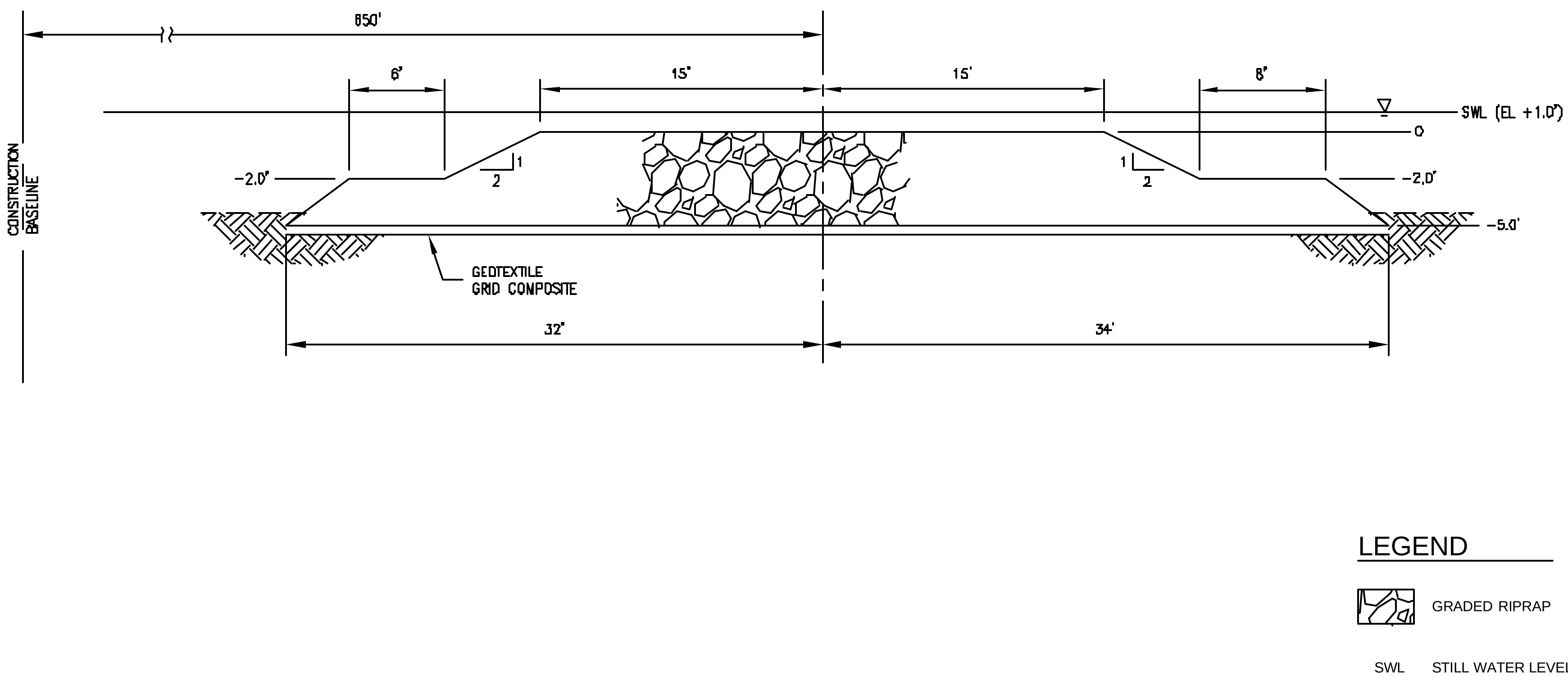
1 C4 C5 EXISTING SITE PLAN

SCALE: 1"=600'

PLAN OF BORINGS AND FIELD VANE SHEAR TESTS
TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON, LOUISIANA

0 300 600

NOTES:
1. APPLIED BEARING PRESSURE BENEATH THE BREAKWATER SHOULD NOT EXCEED THE ALLOWABLE BEARING CAPACITY
2. SEE TEXT FOR A DISCUSSION ON SLOPE AND BASE STABILITY OF THE BREAKWATER.
3. SEE TEXT FOR A DISCUSSION ON GEOFABRIC/GEOGRID COMPOSITE.



TYPICAL PROPOSED CROSS-SECTION - ROCK REEF BREAKWATER

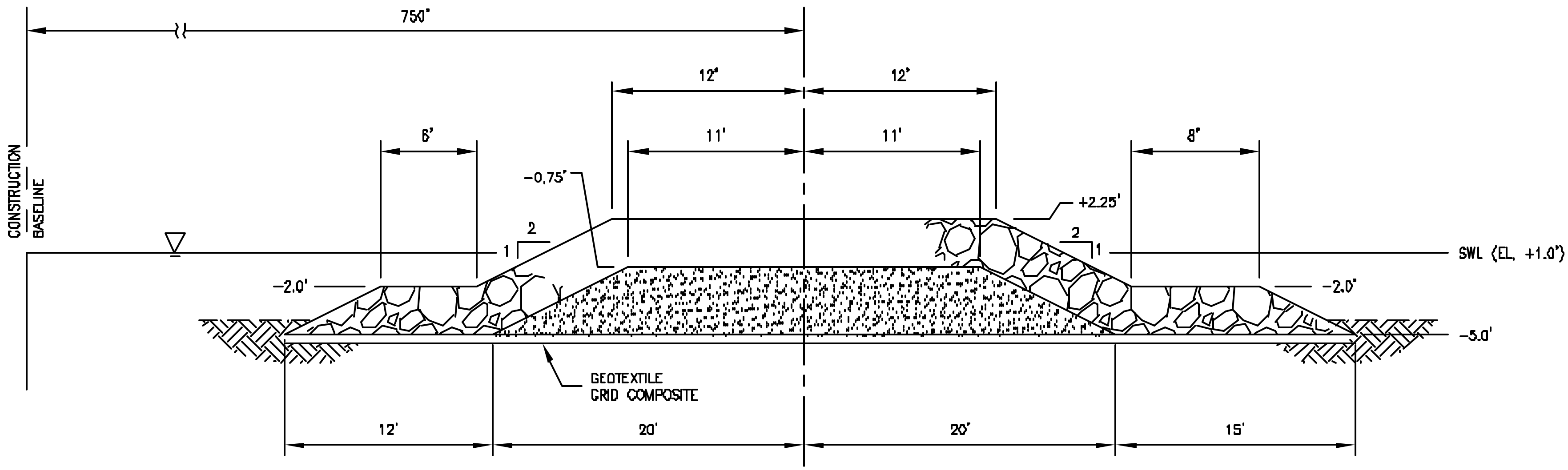
TEST SECTIONS - ROCKEFELLER REFUGE

GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

(NOT TO SCALE)

NOTES:
1. APPLIED BEARING PRESSURE BENEATH THE BREAKWATER SHOULD NOT EXCEED THE ALLOWABLE BEARING CAPACITY
2. SEE TEXT FOR A DISCUSSION ON SLOPE AND BASE STABILITY OF THE BREAKWATER.
3. LIGHTWEIGHT AGGREGATE SHALL BE CONTAINED IN GEOTUBES OR GEOBAGS PROVIDING A SIMILAR FUNCTION.
4. SEE TEXT FOR A DISCUSSION ON GEOFABRIC/GEOGRID COMPOSITE.



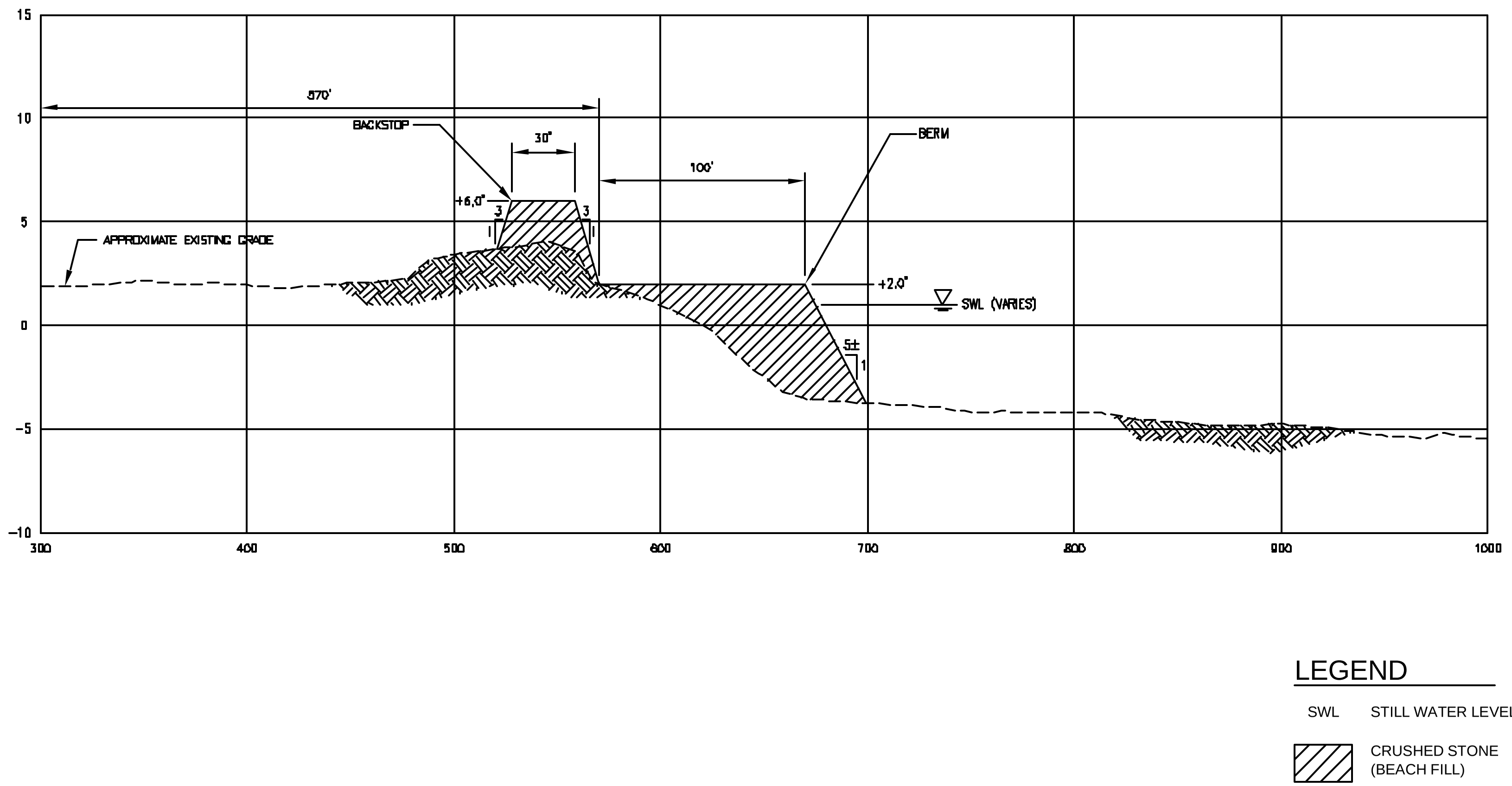
LEGEND

(hatch)	LIGHTWEIGHT AGGREGATE (SEE NOTE 3)
(hatch)	GRADED RIPRAP
SWL	STILL WATER LEVEL

TYPICAL PROPOSED CROSS-SECTION - ROCK REEF BREAKWATER WITH LIGHTWEIGHT AGGREGATE

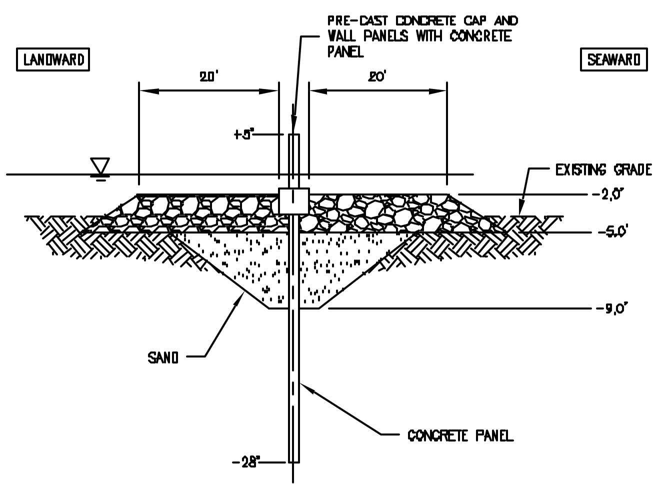
TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA
(NOT TO SCALE)

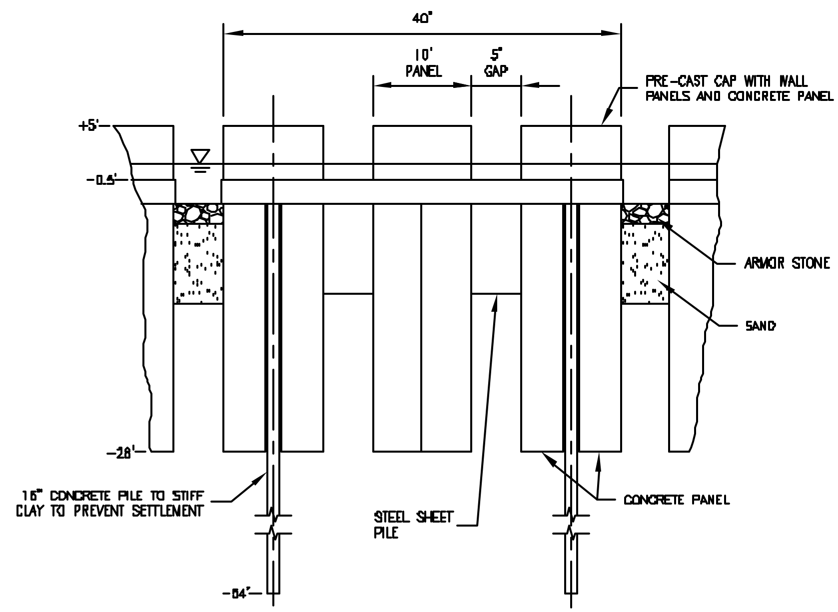
NOTES:
1. APPLIED BEARING PRESSURE BENEATH THE BEACH FILL SHOULD NOT EXCEED THE ALLOWABLE BEARING CAPACITY
2. BACKSTOP AND BERM SIDE SLOPES SHOULD BE FLATTENED AS RECOMMENDED IN THE TEXT.
3. STABILITY OF THE BEACH FILL COULD BE IMPROVED BY UTILIZING A GEOFABRIC/GEOGRID COMPOSITE AS DESCRIBED IN THE TEXT.



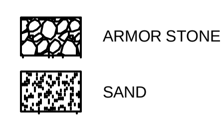
TYPICAL PROPOSED CROSS-SECTION - BEACH FILL
TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA
(NOT TO SCALE)

NOTES:
1. APPLIED BEARING PRESSURE BENEATH THE SAND FILL SHOULD NOT EXCEED THE ALLOWABLE BEARING CAPACITY
2. SIDE SLOPES OF THE EXCAVATION FOR THE PLACEMENT OF THE SAND BACKFILL SHOULD BE FLATTENED AS DISCUSSED IN TEXT.
3. REFER TO PLATE 35 FOR MINIMUM LATERAL EXTENT OF SAND AS DISCUSSED IN TEXT.
4. DRIVEN CONCRETE PILES SHOULD BE DESIGNED FOR A FACTOR OF SAFTEY OF 2 FOR SUSTAINED COMPRESSIVE AND TRANSIENT TENSILE LOADS.
5. THE EMBEDMENT OF CONCRETE PANELS SHOULD BE BASED ON AN ADEQUATE FOCTOR OF SAFTEY FOR BOTH INTERNAL AND GLOBAL STABILITY FAILURES MODES.





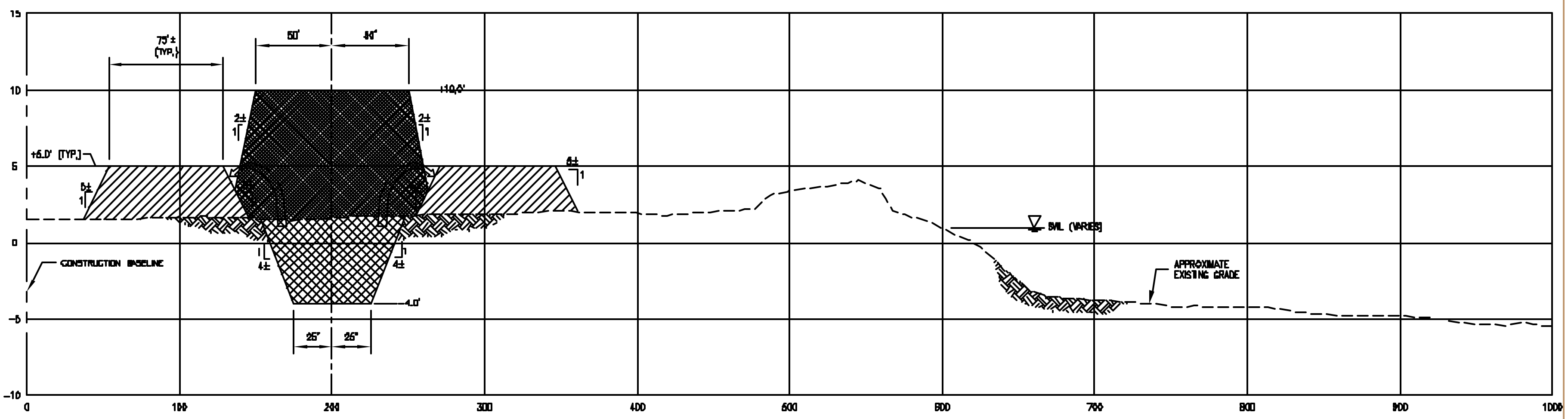
LEGEND

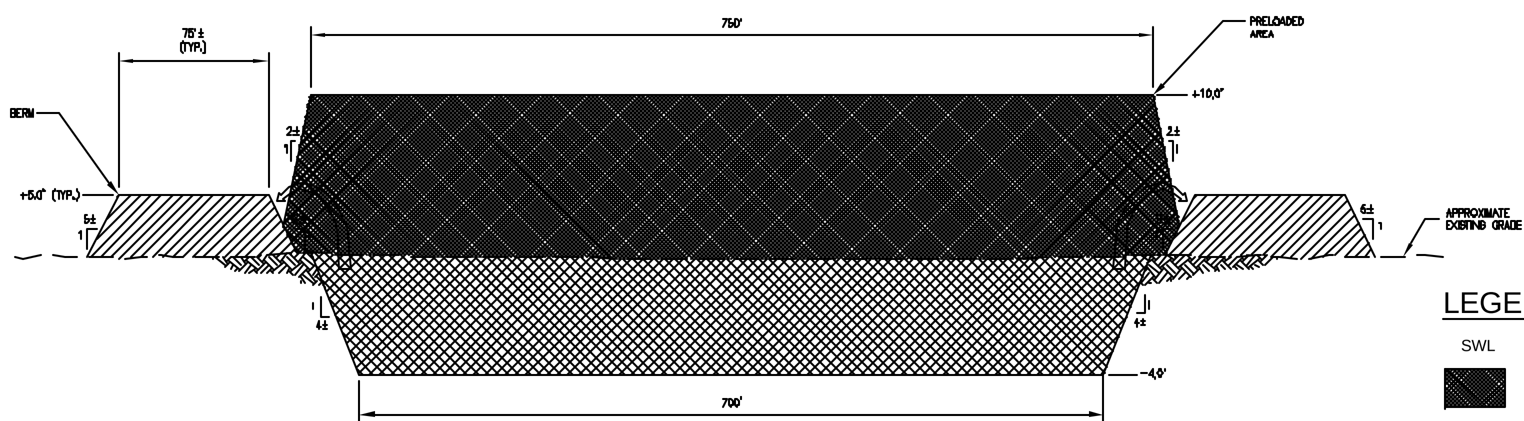


TYPICAL PROPOSED CROSS-SECTION - CONCRETE PANEL BREAKWATER
TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA
(NOT TO SCALE)

NOTES:

1. BUILDING THE PRELOADING AREA AS PLANNED WILL LIKELY CAUSE A BEARING CAPACITY FAILURE AS DISCUSSED IN THE TEXT.
2. SIDE SLOPES OF THE EXCAVATION FOR FILL PLACEMENT SHOULD BE FLATTENED AS DISCUSSED IN THE TEXT.
3. SEE TEXT FOR RECOMMENDATIONS ON MINIMUM DISTANCE BETWEEN THE BERM AND THE EXCAVATION.
4. PROPER TECHNIQUE OF PRELOADING AND BUILDING THE AREA IN STAGES IS DICUSSED IN THE TEXT.



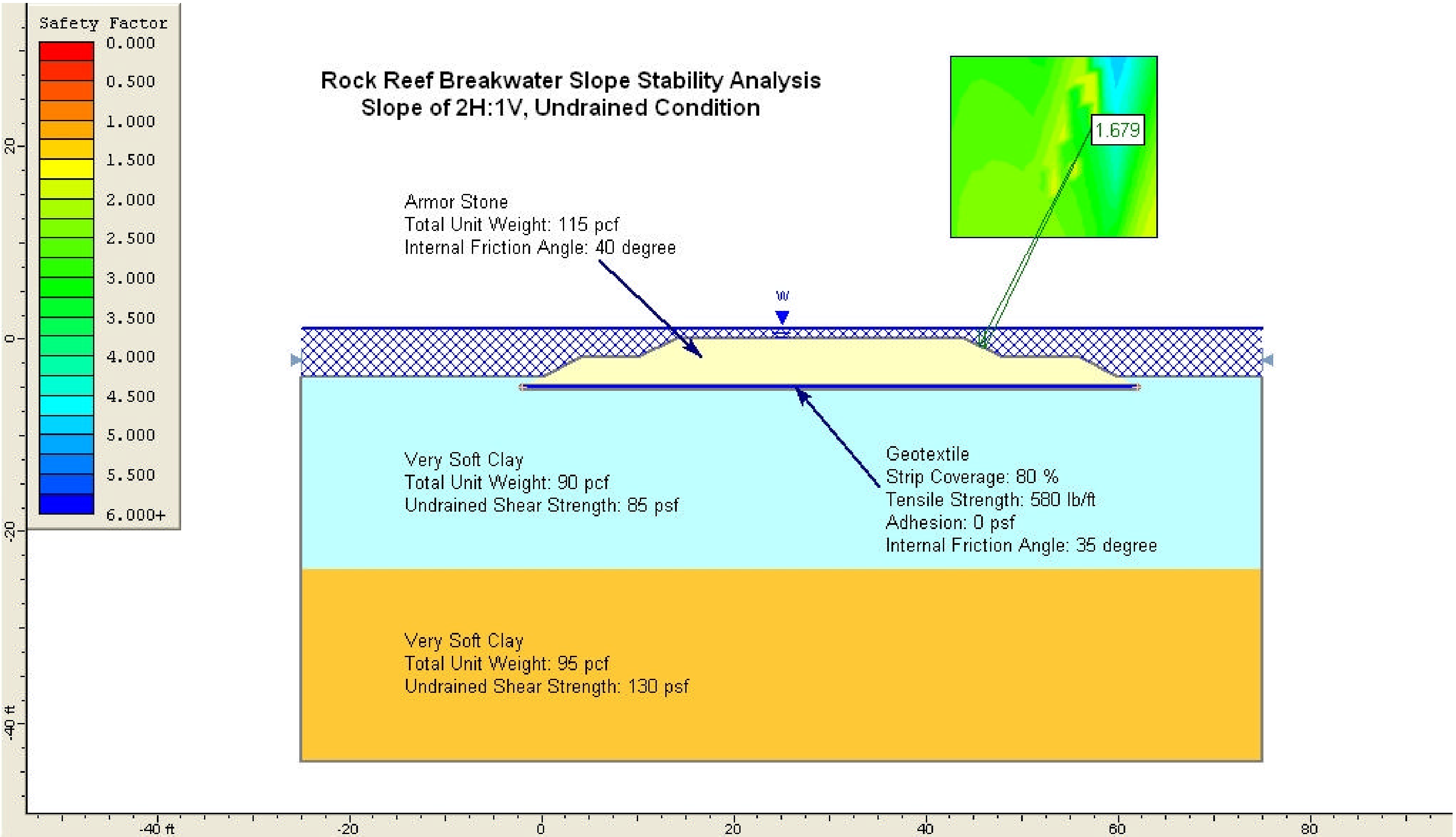


LEGEND

SWL	STILL WATER LEVEL
	IMPORTED CLAY FILL MATERIAL
	EXCAVATE AND BACKFILL WITH IMPORTED CLAY FILL MATERIAL
	SIDECAST IN-SITU MATERIAL

TYPICAL PROPOSED CROSS-SECTION - PRE-LOADING TEST SECTION

TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA
(NOT TO SCALE)

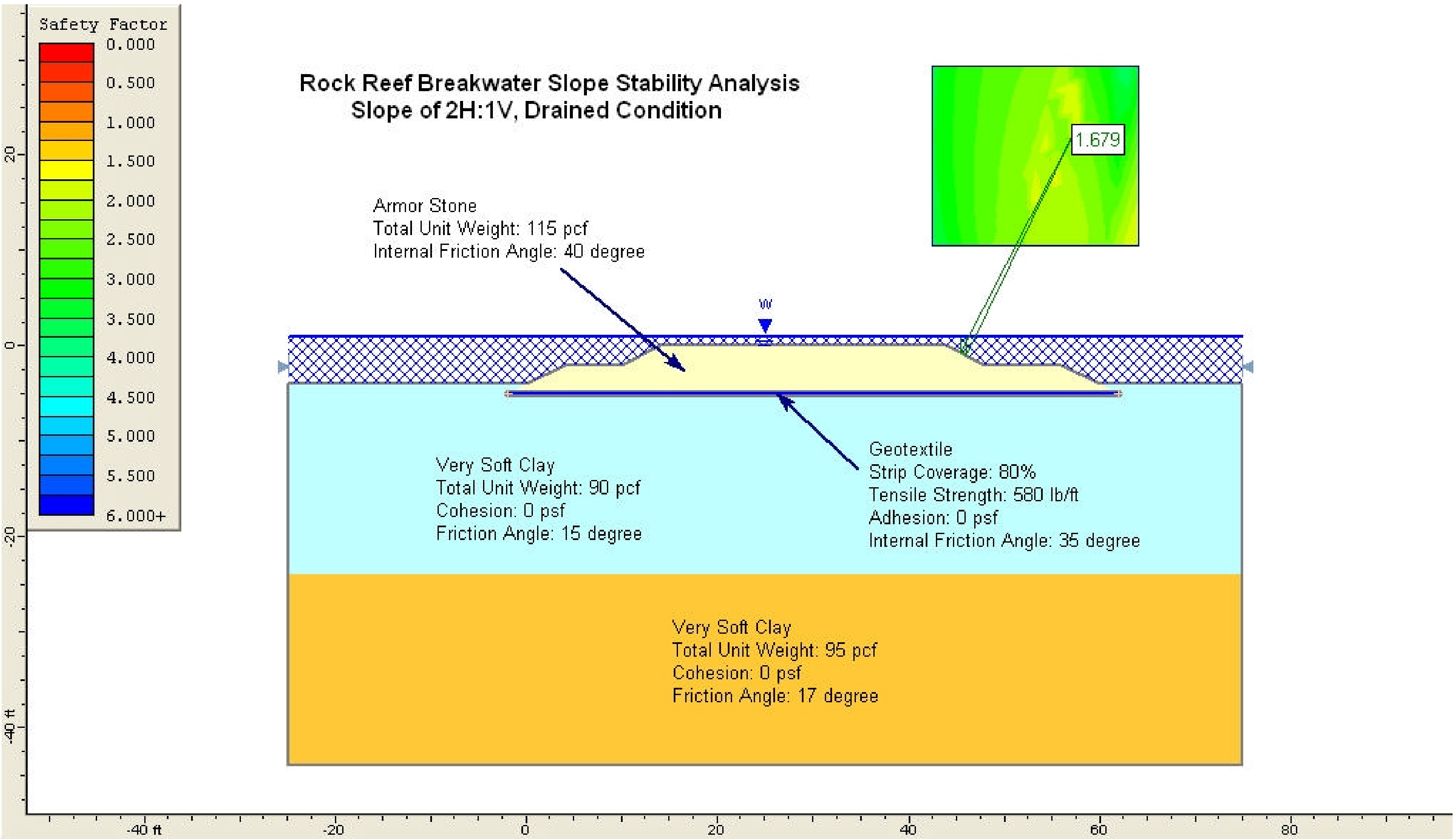


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE

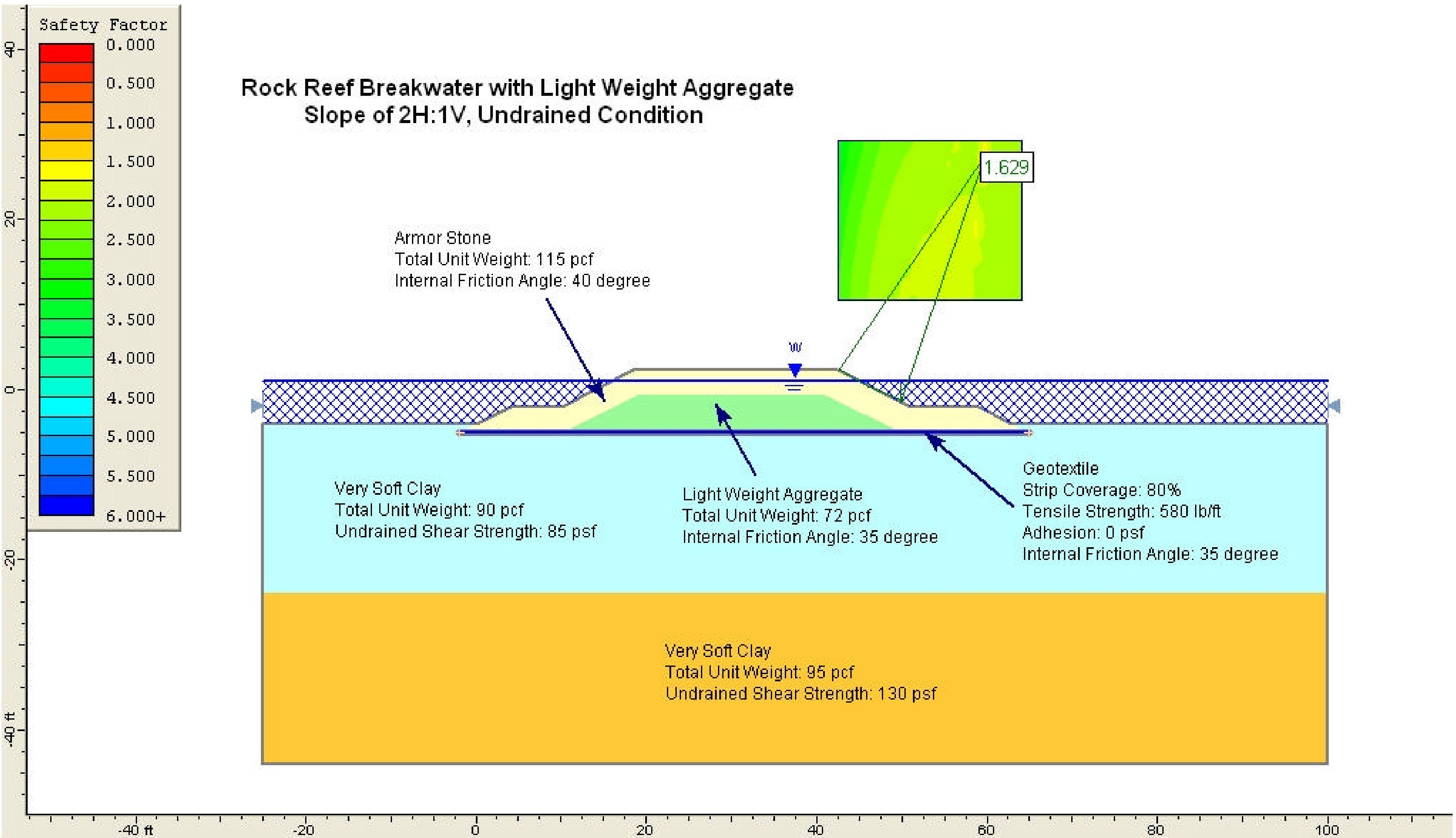
GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

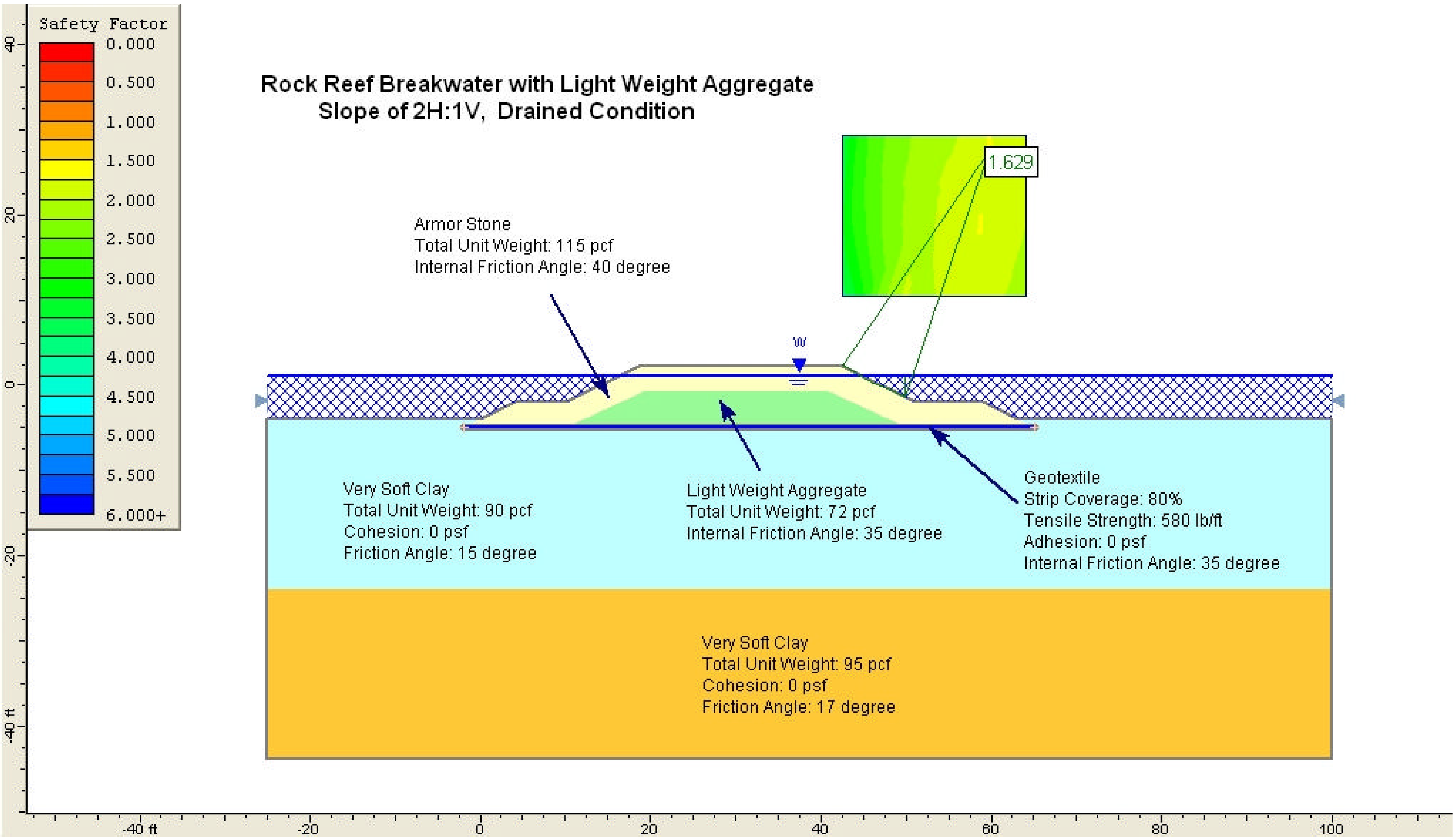


RESULTS OF STABILITY ANALYSIS

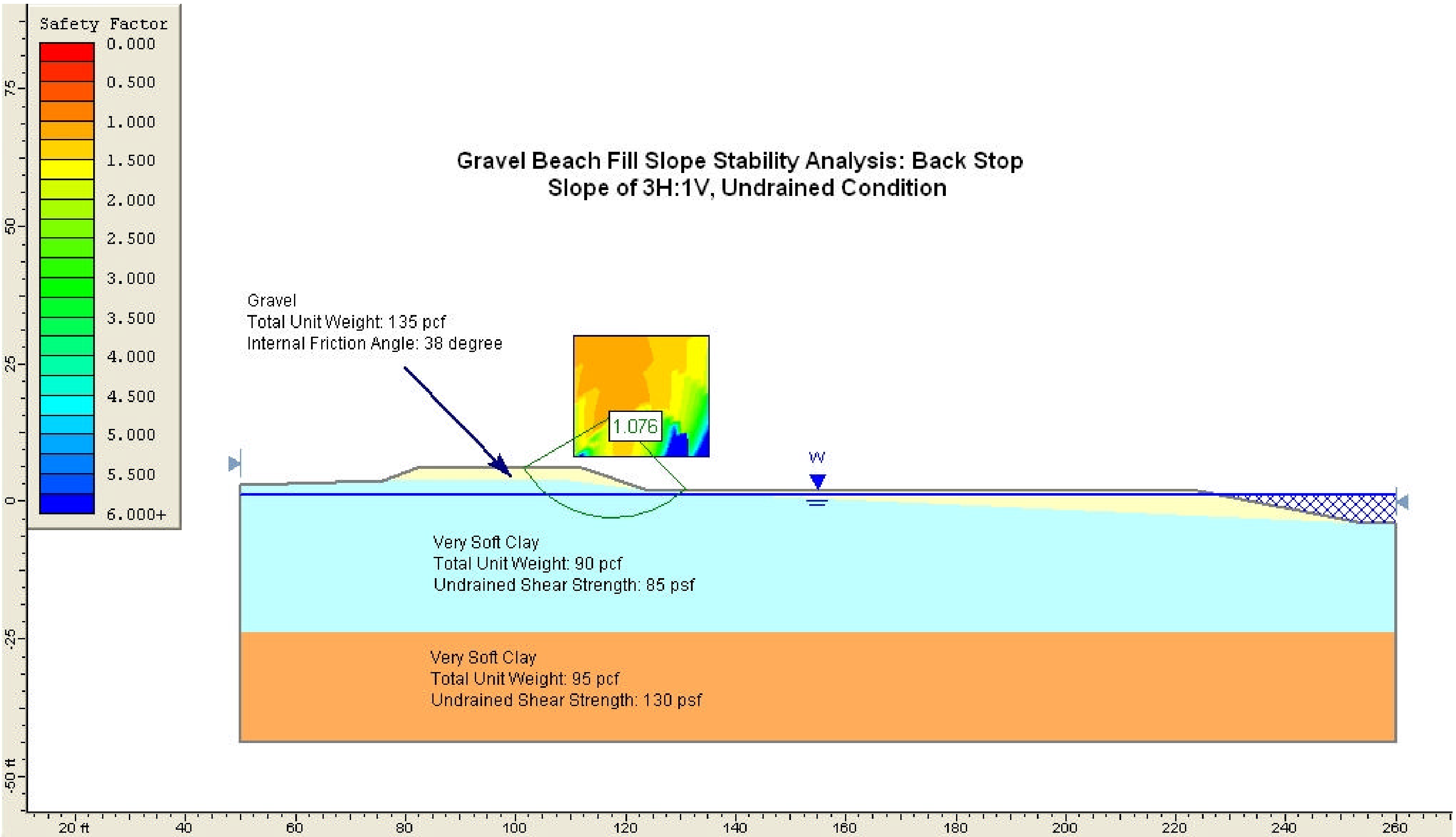
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



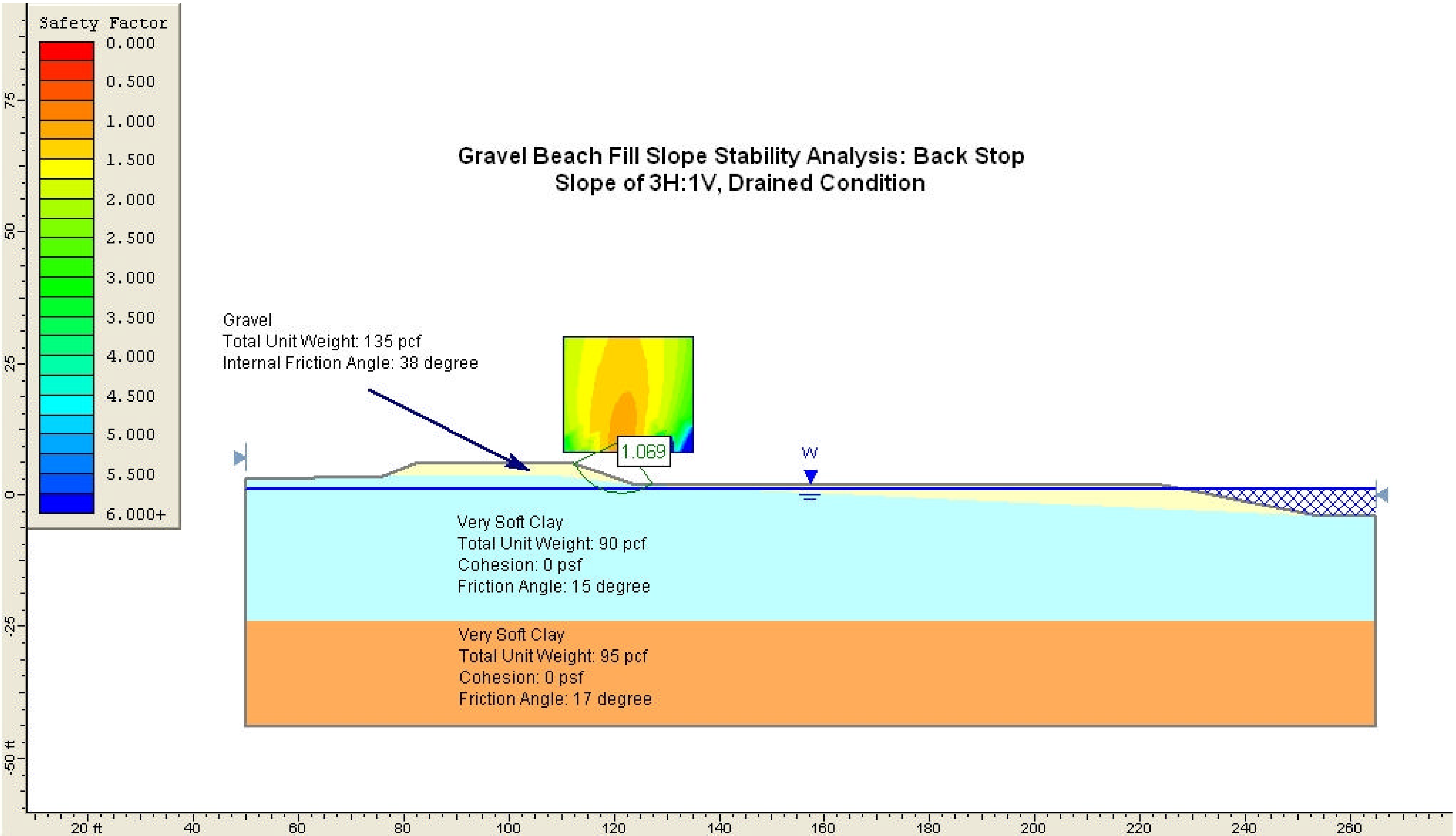
RESULTS OF STABILITY ANALYSIS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



RESULTS OF STABILITY ANALYSIS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



RESULTS OF STABILITY ANALYSIS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

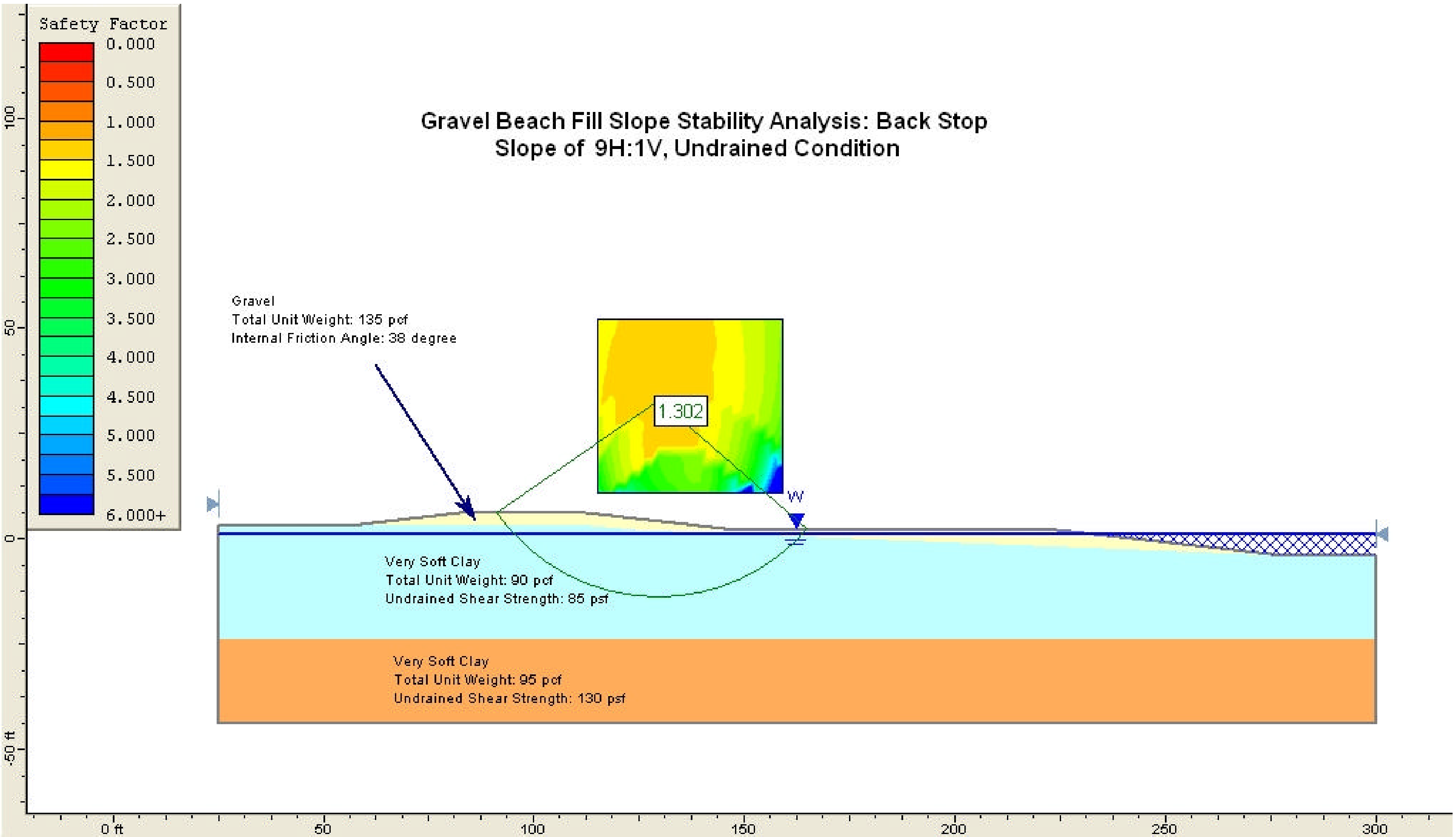


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE

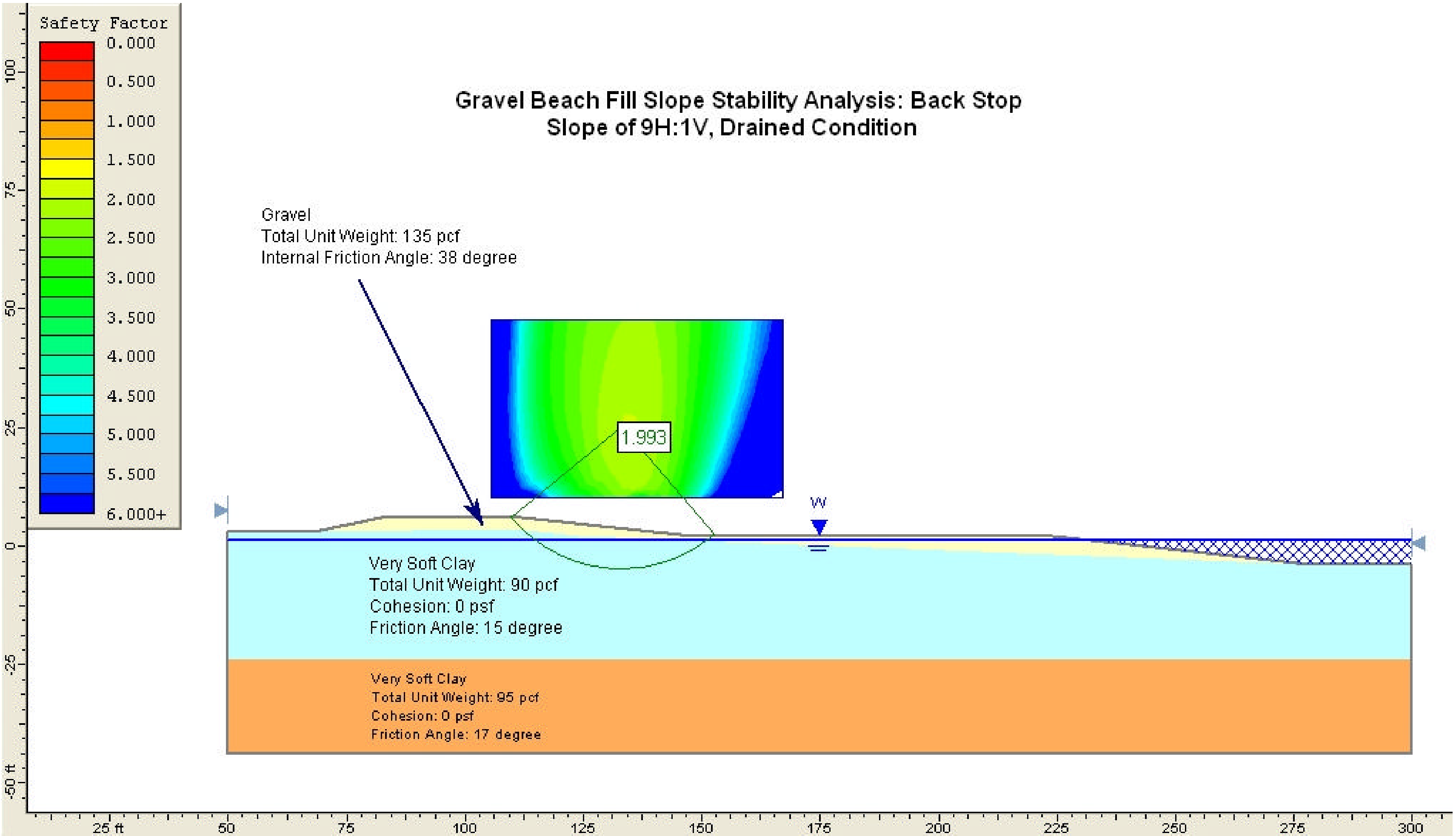
GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

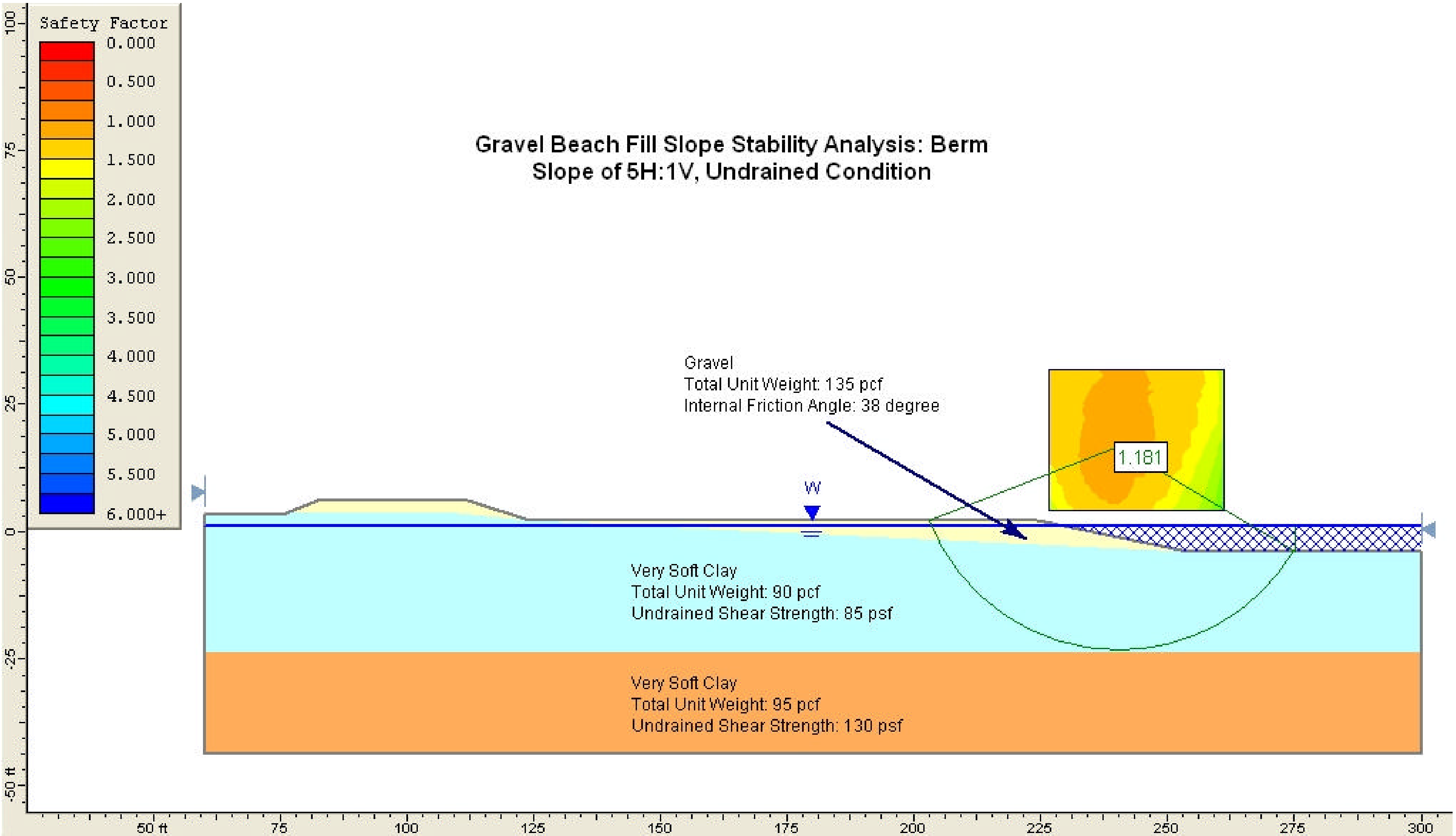


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



RESULTS OF STABILITY ANALYSIS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



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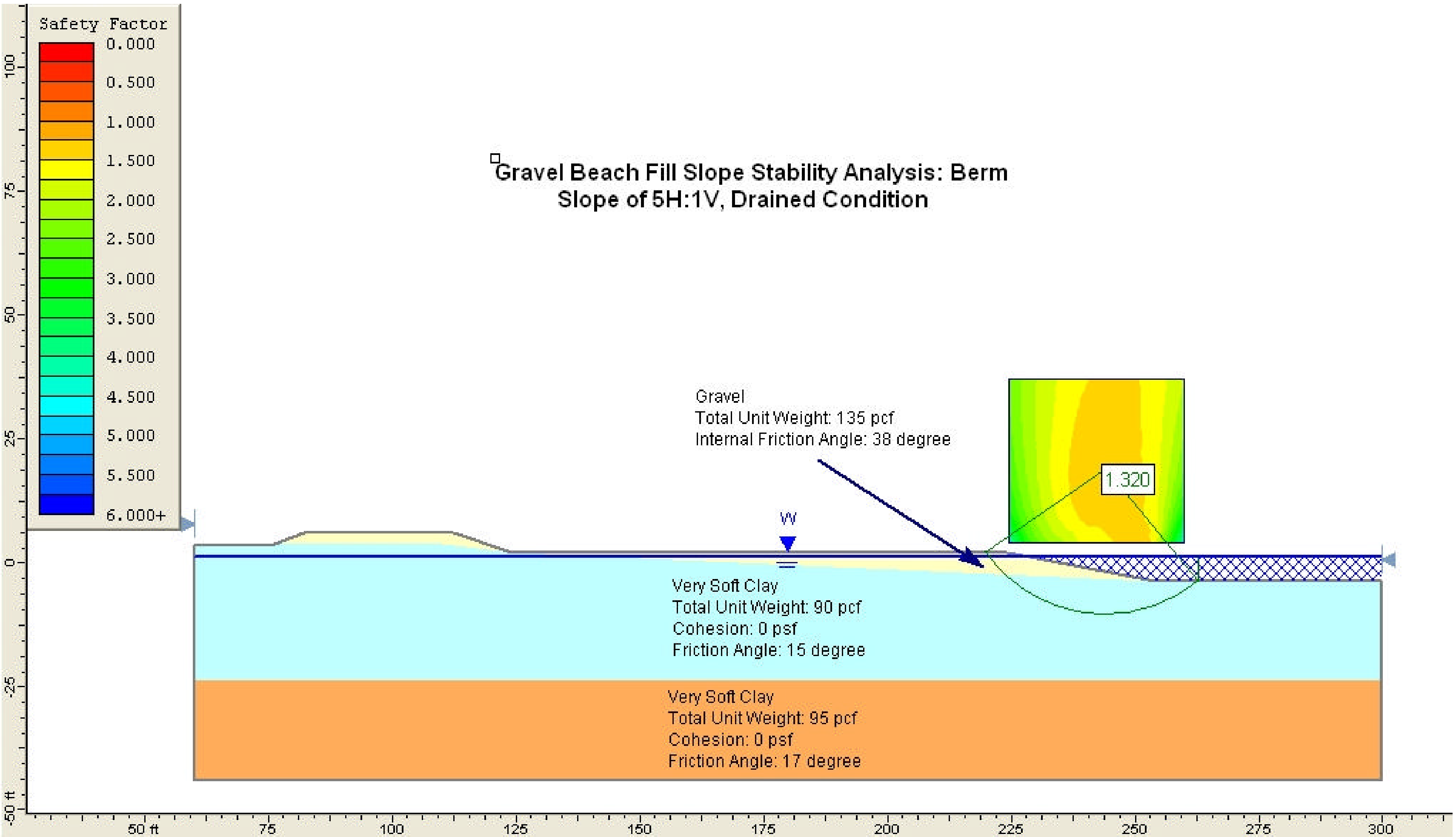
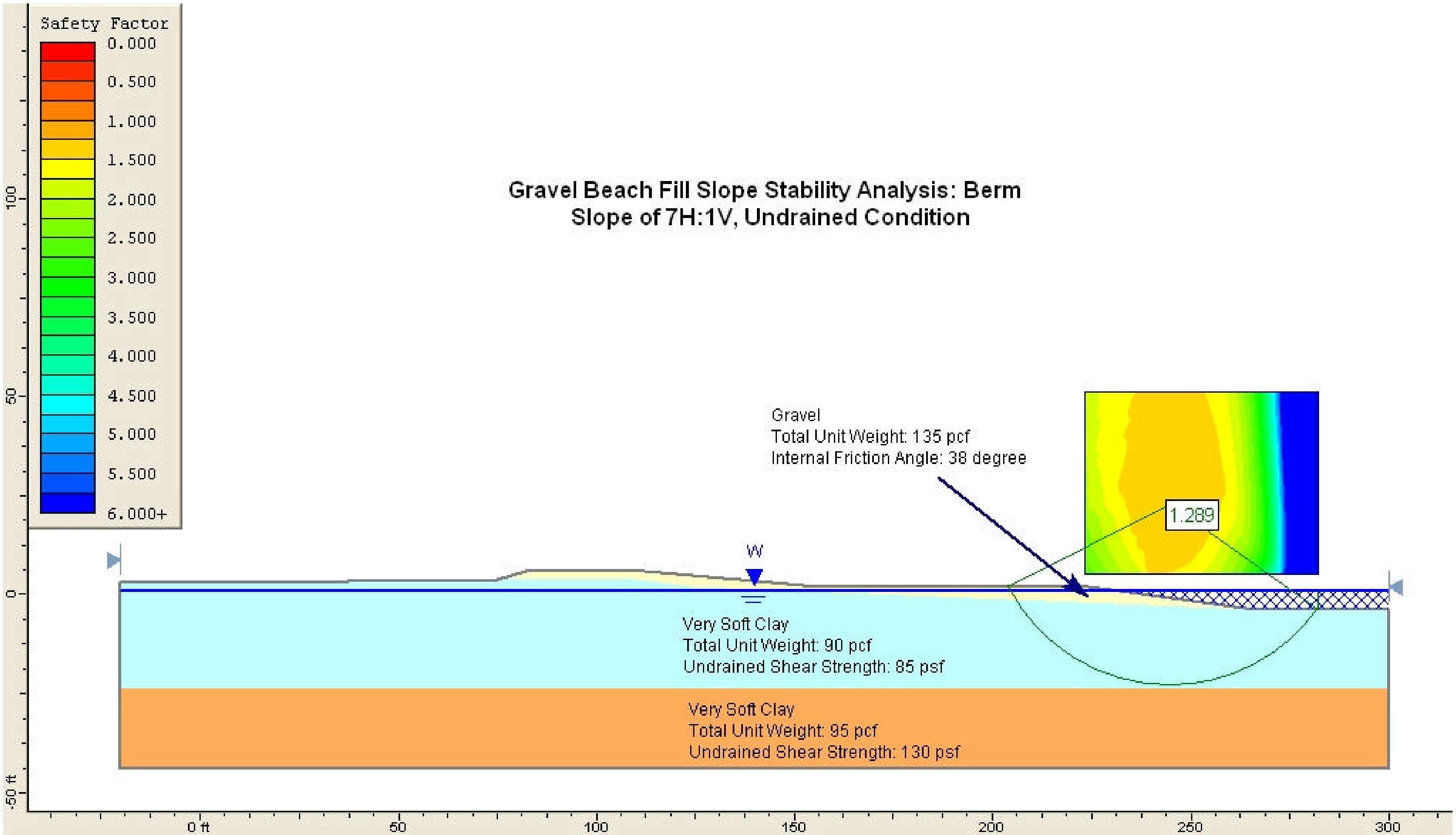
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



RESULTS OF STABILITY ANALYSIS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

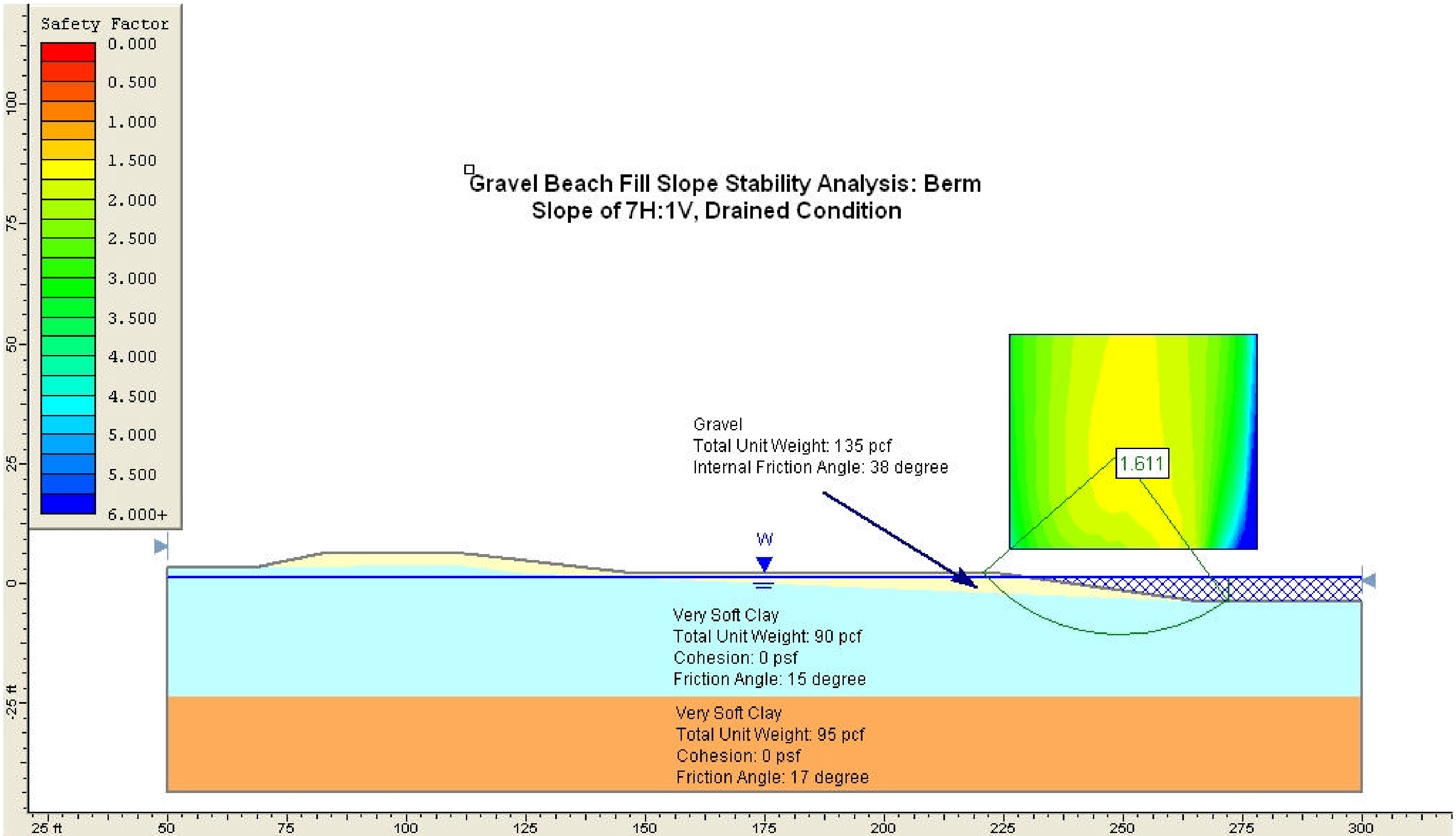


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE

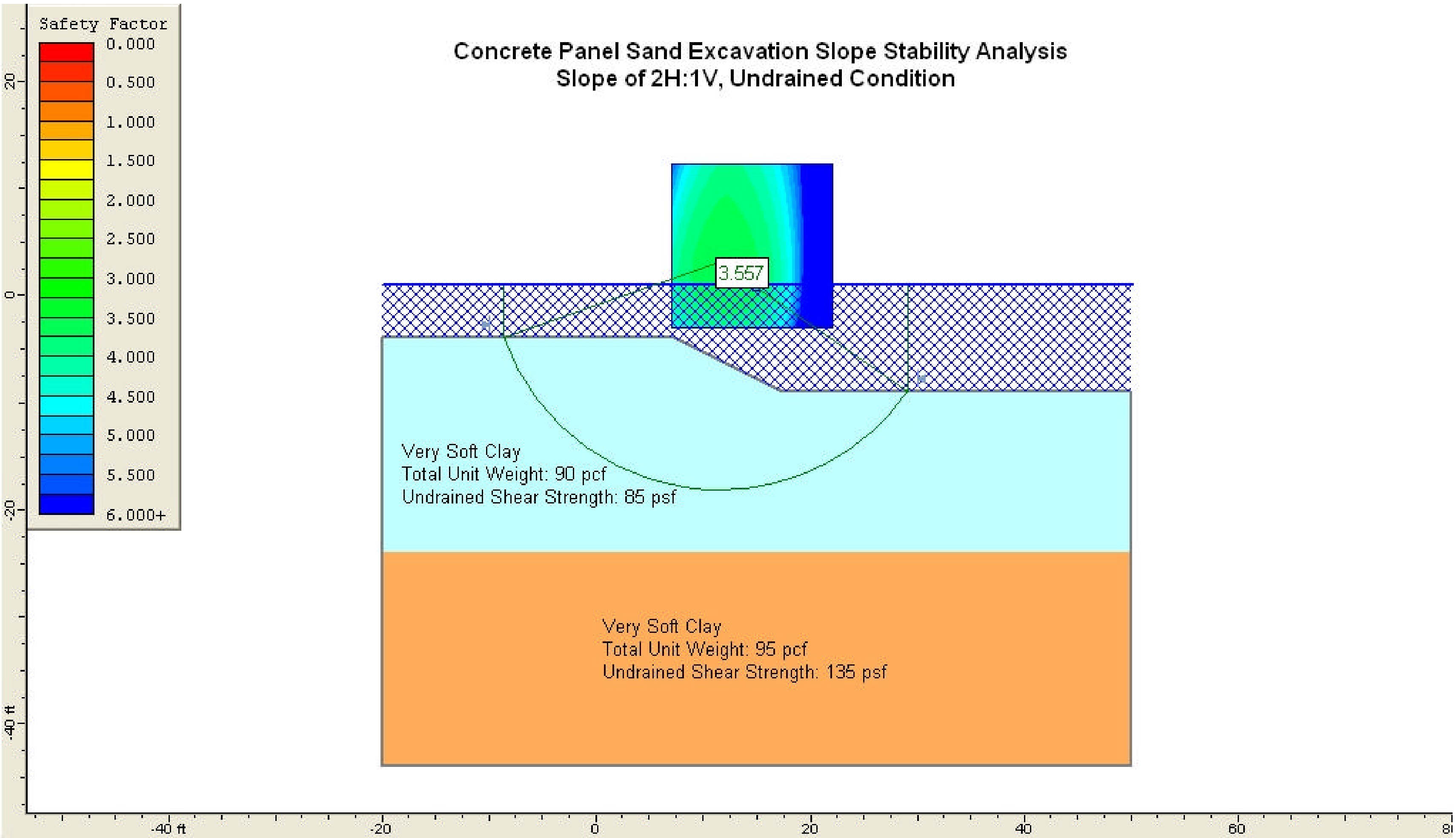
GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA



RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

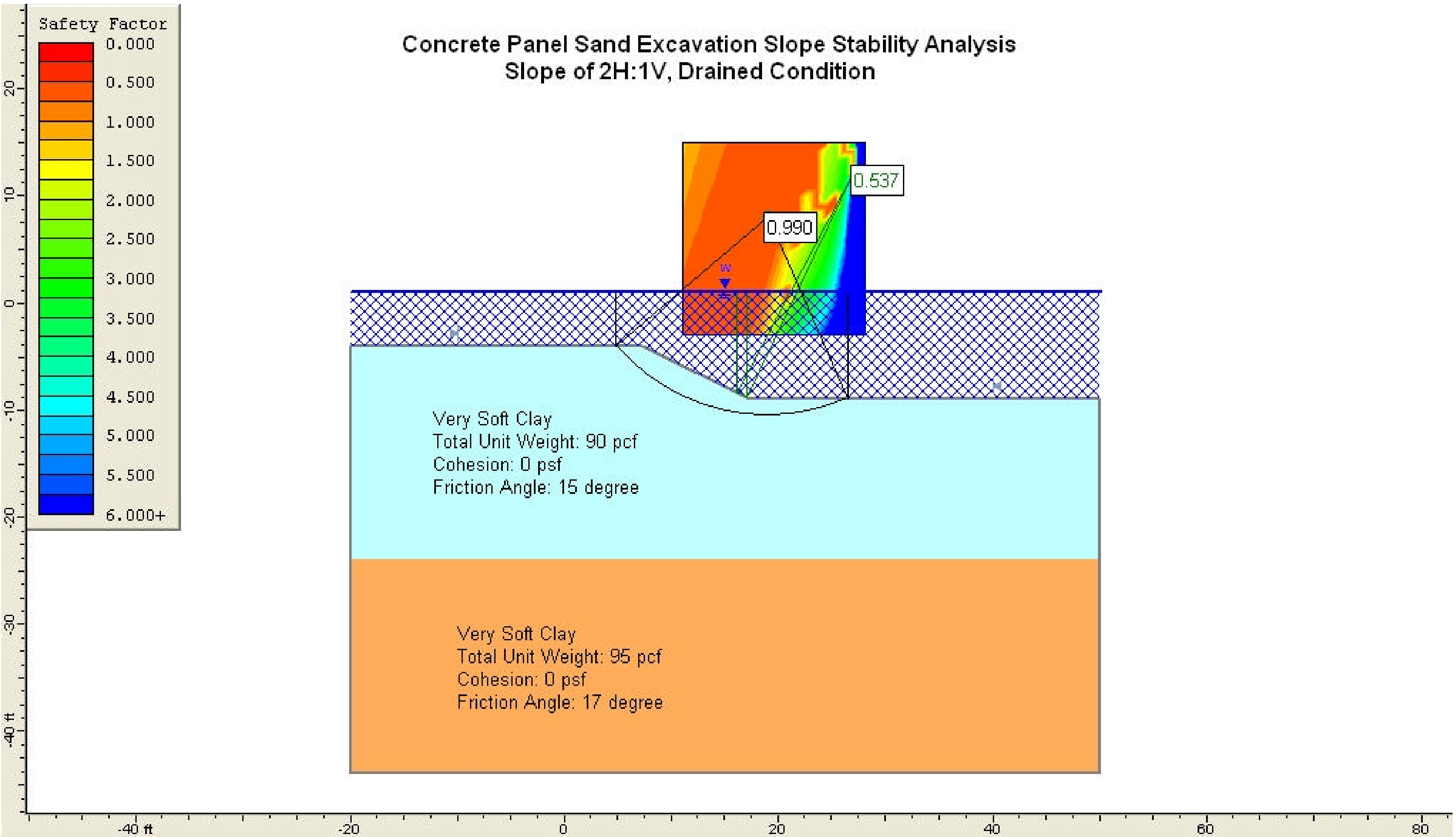


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE

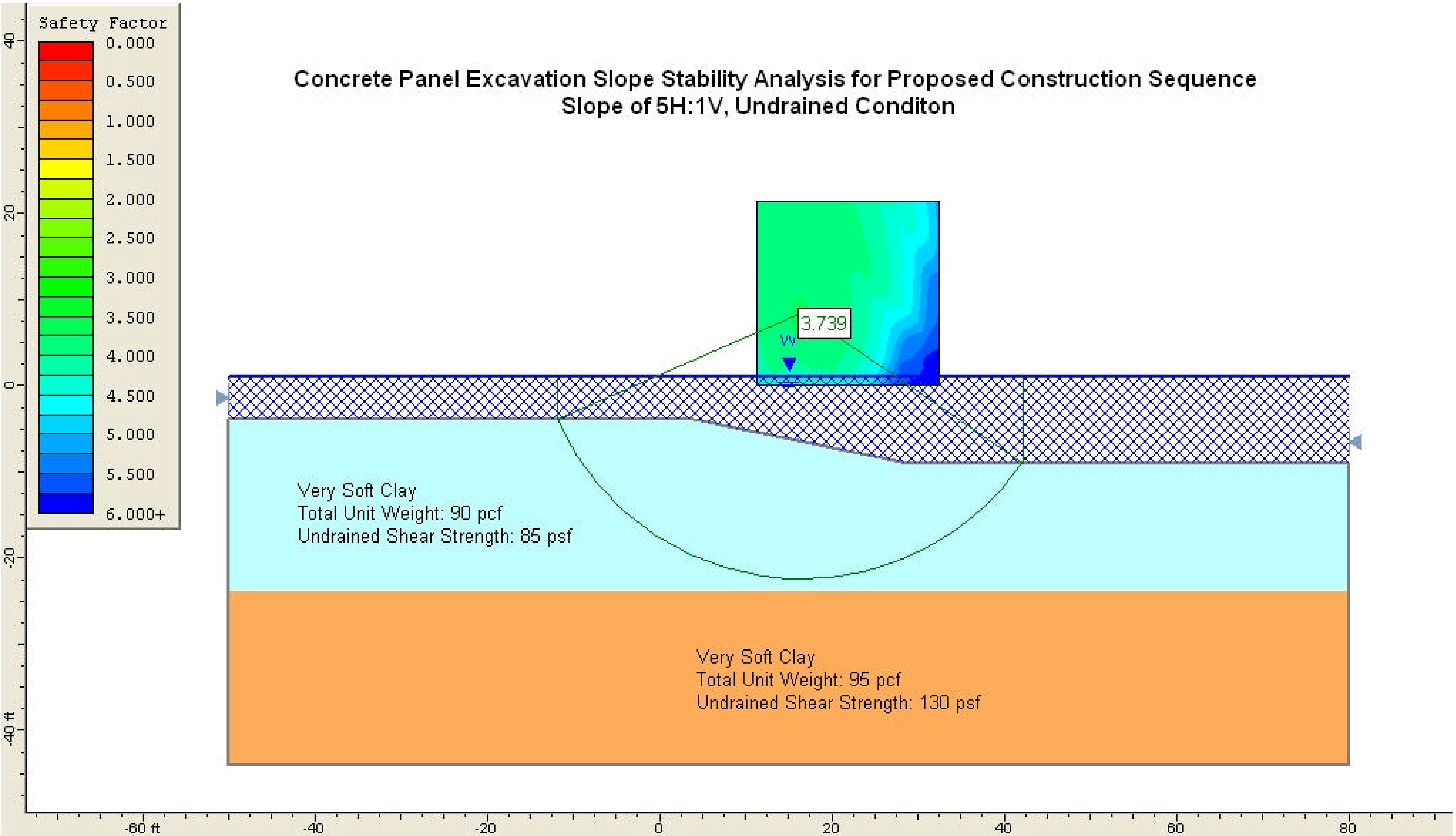
GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA



RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

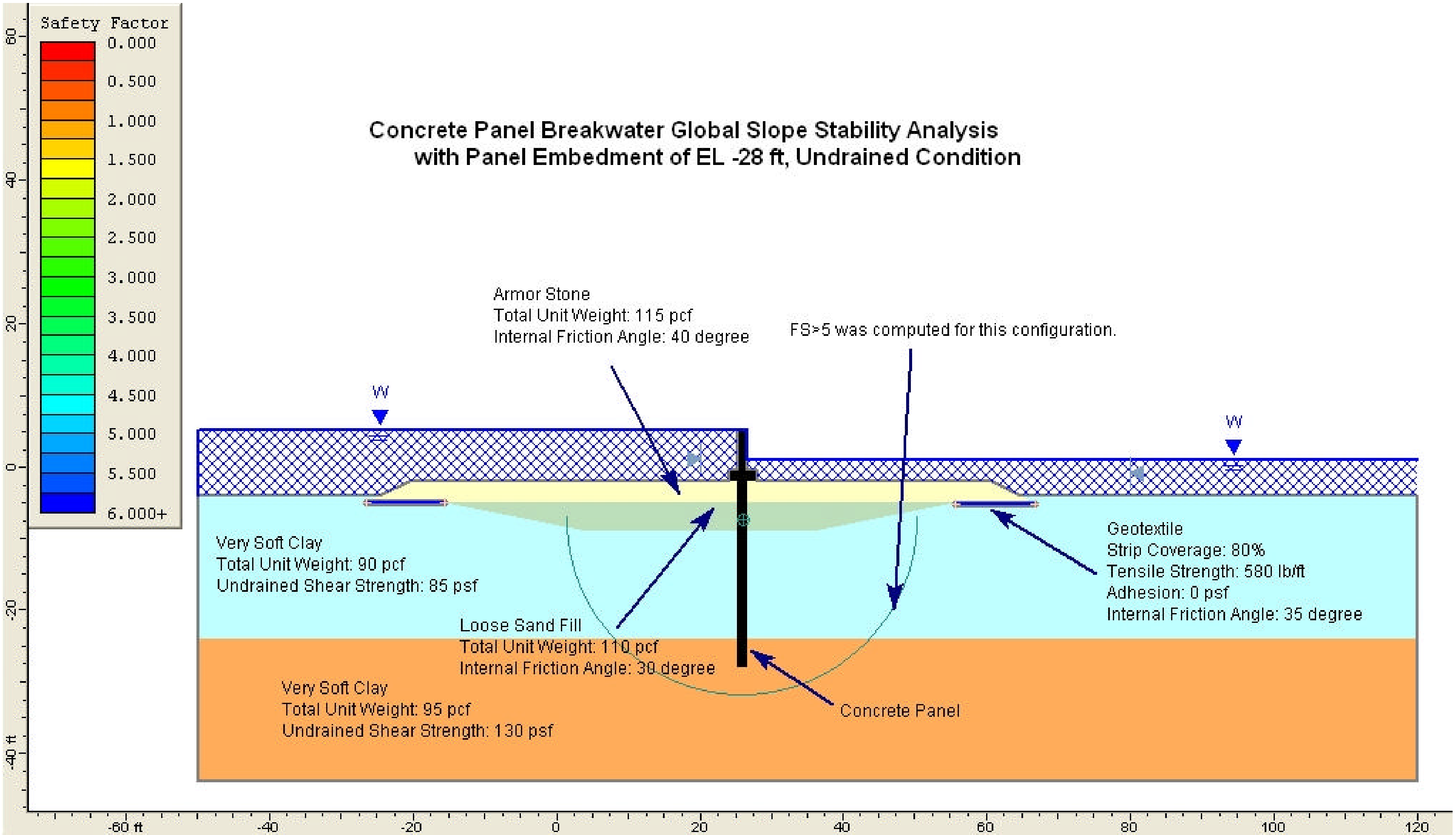


RESULTS OF STABILITY ANALYSIS

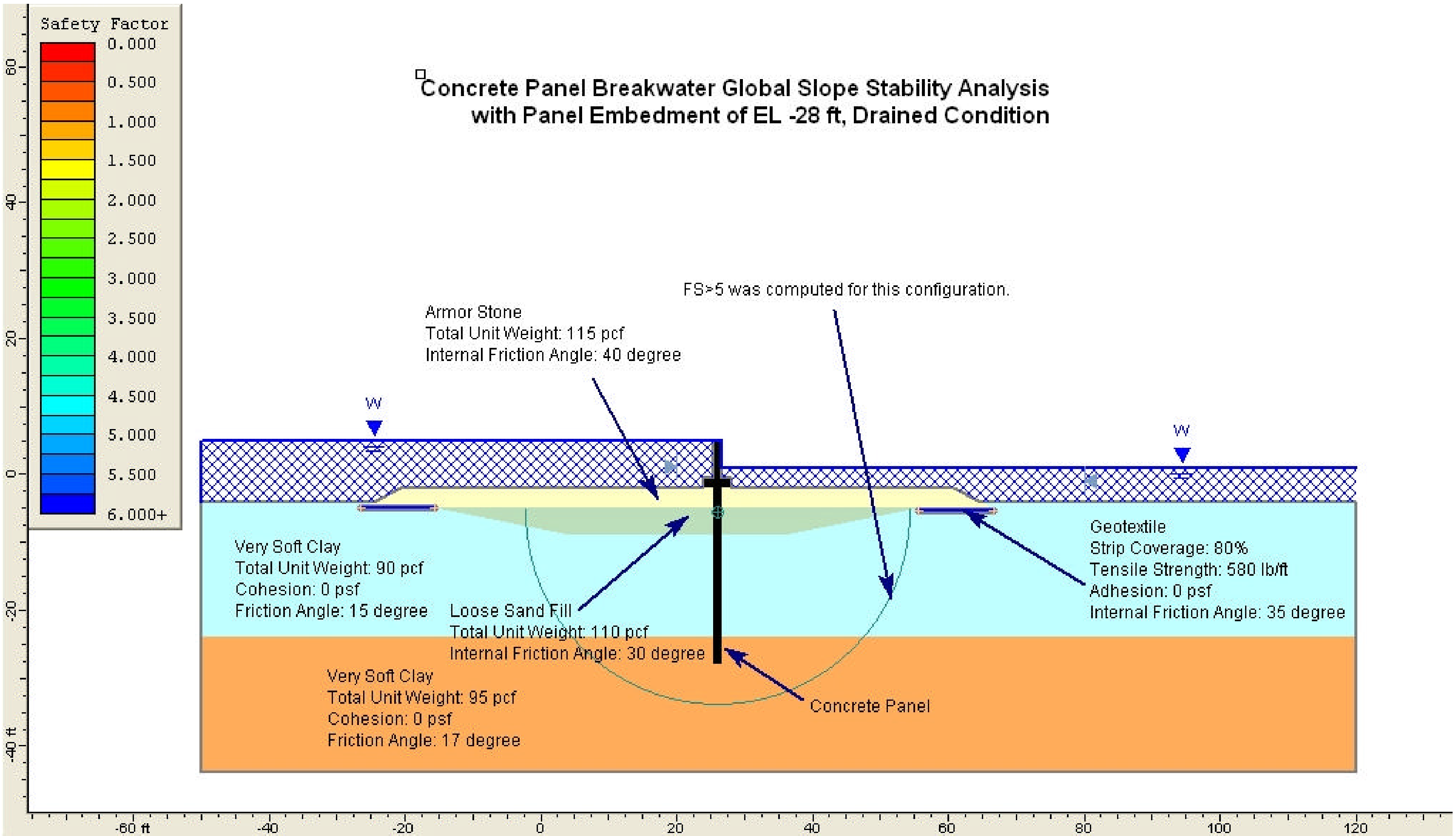
TEST SECTIONS – ROCKEFELLER REFUGE

GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

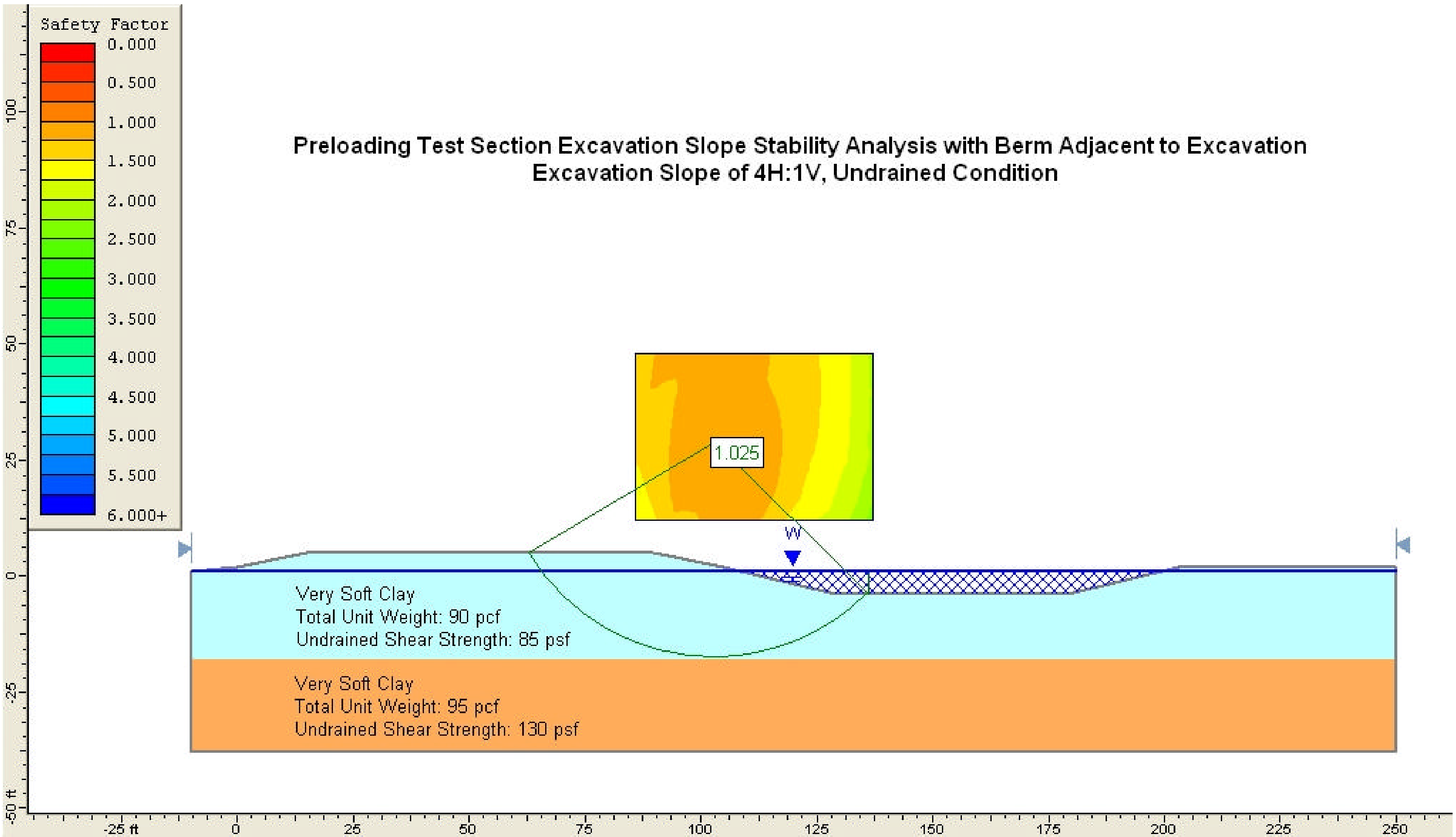


RESULTS OF STABILITY ANALYSIS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



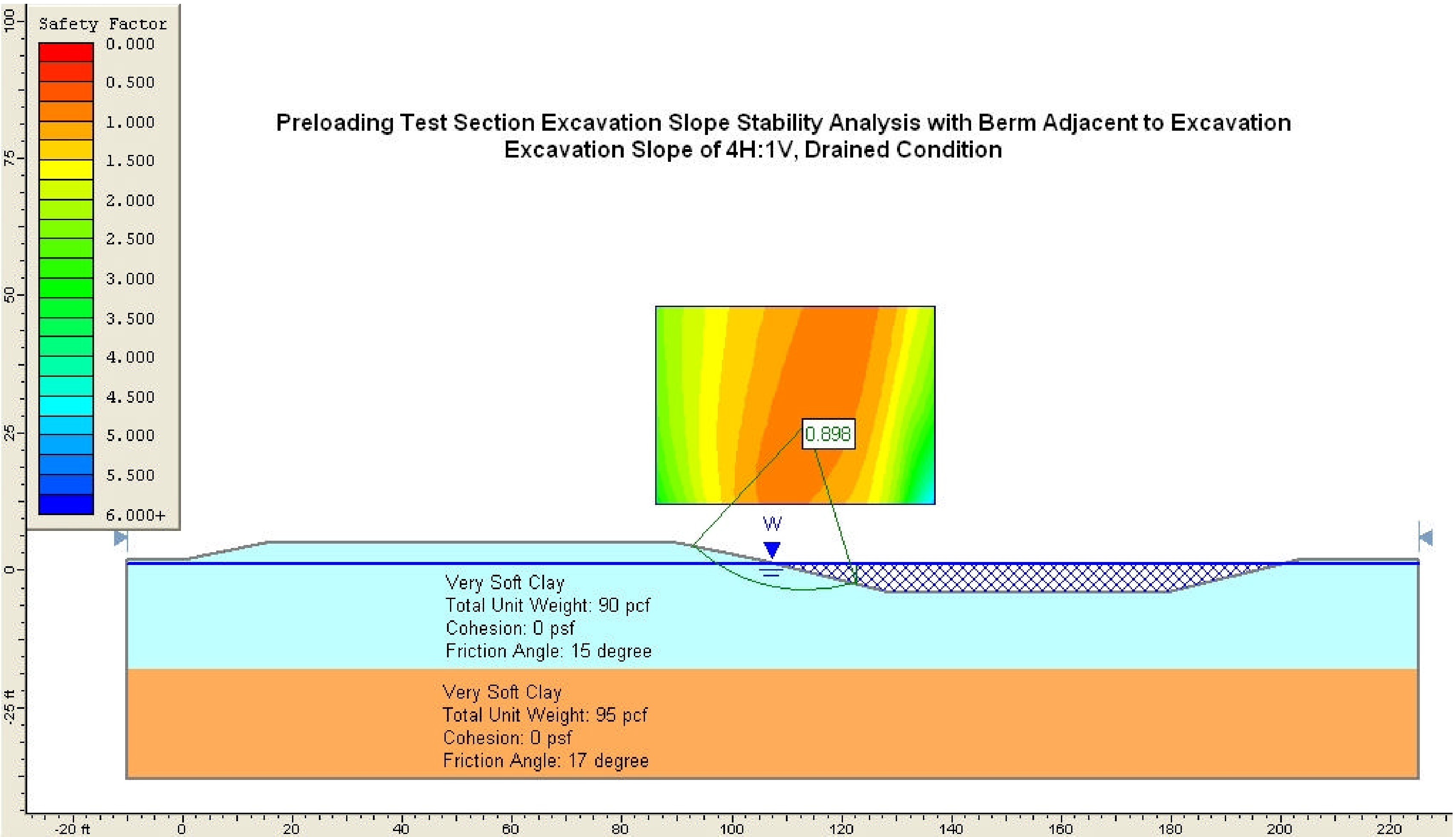
RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

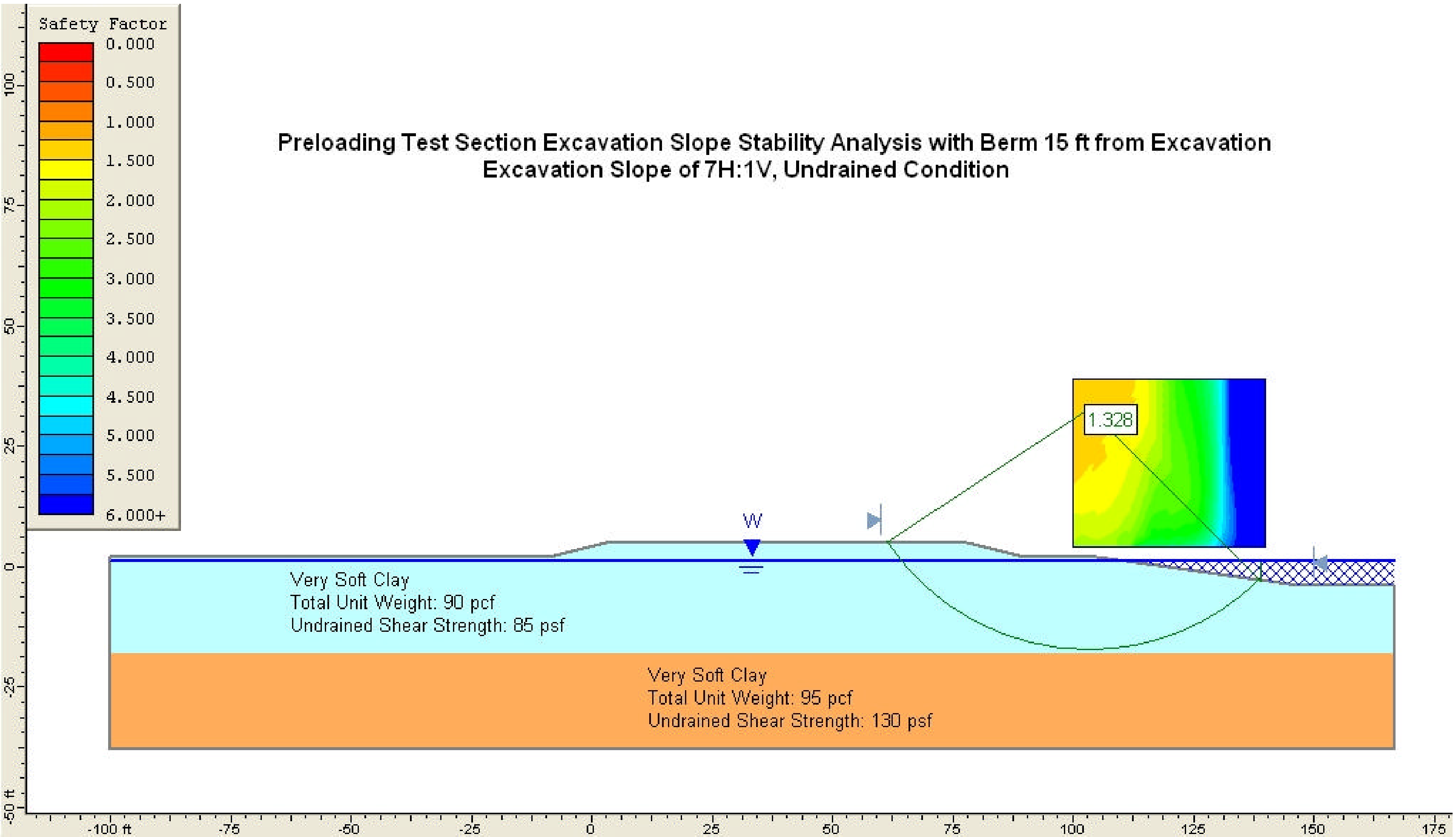


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE

GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

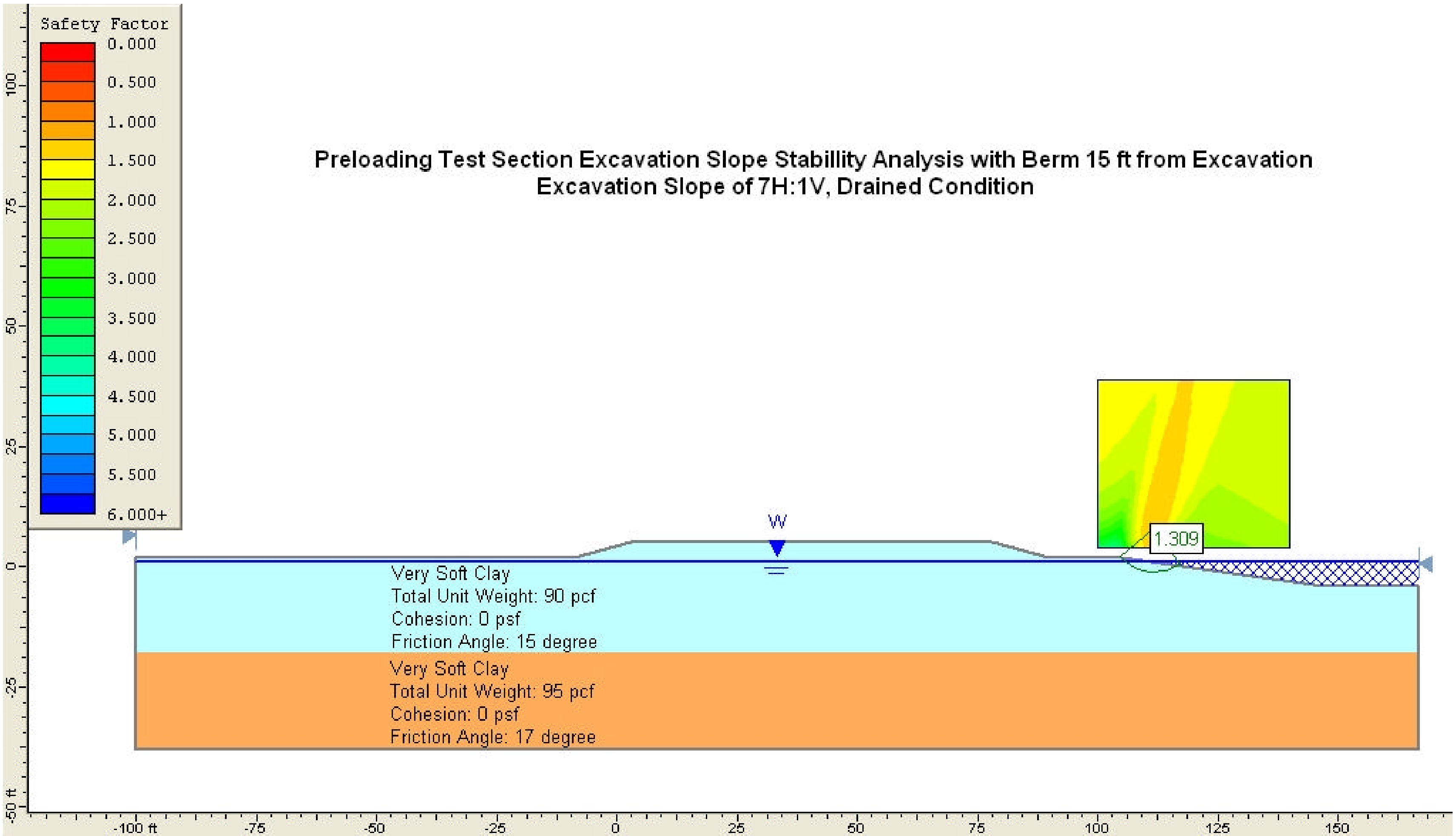


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE

GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

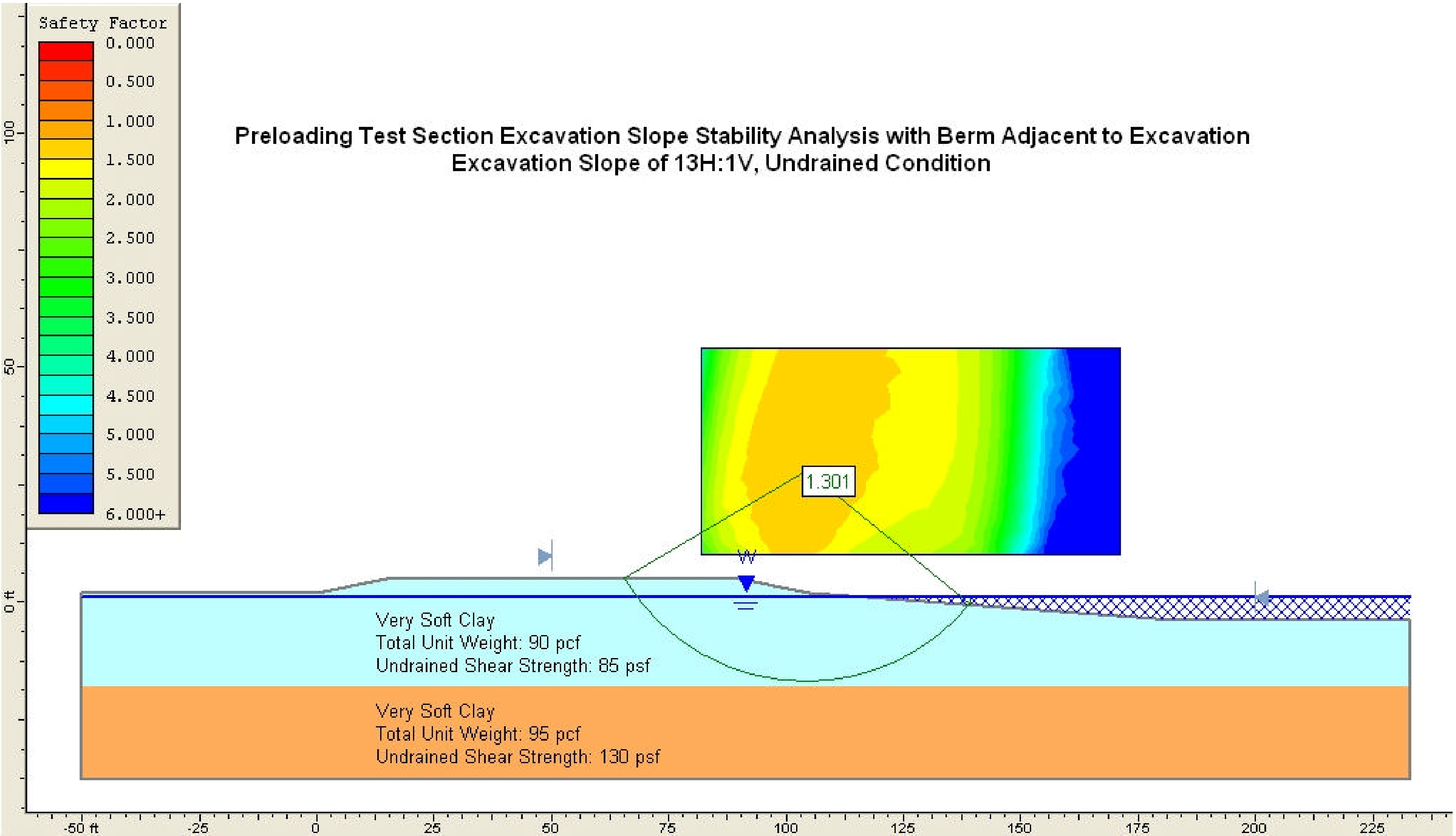


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE

GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

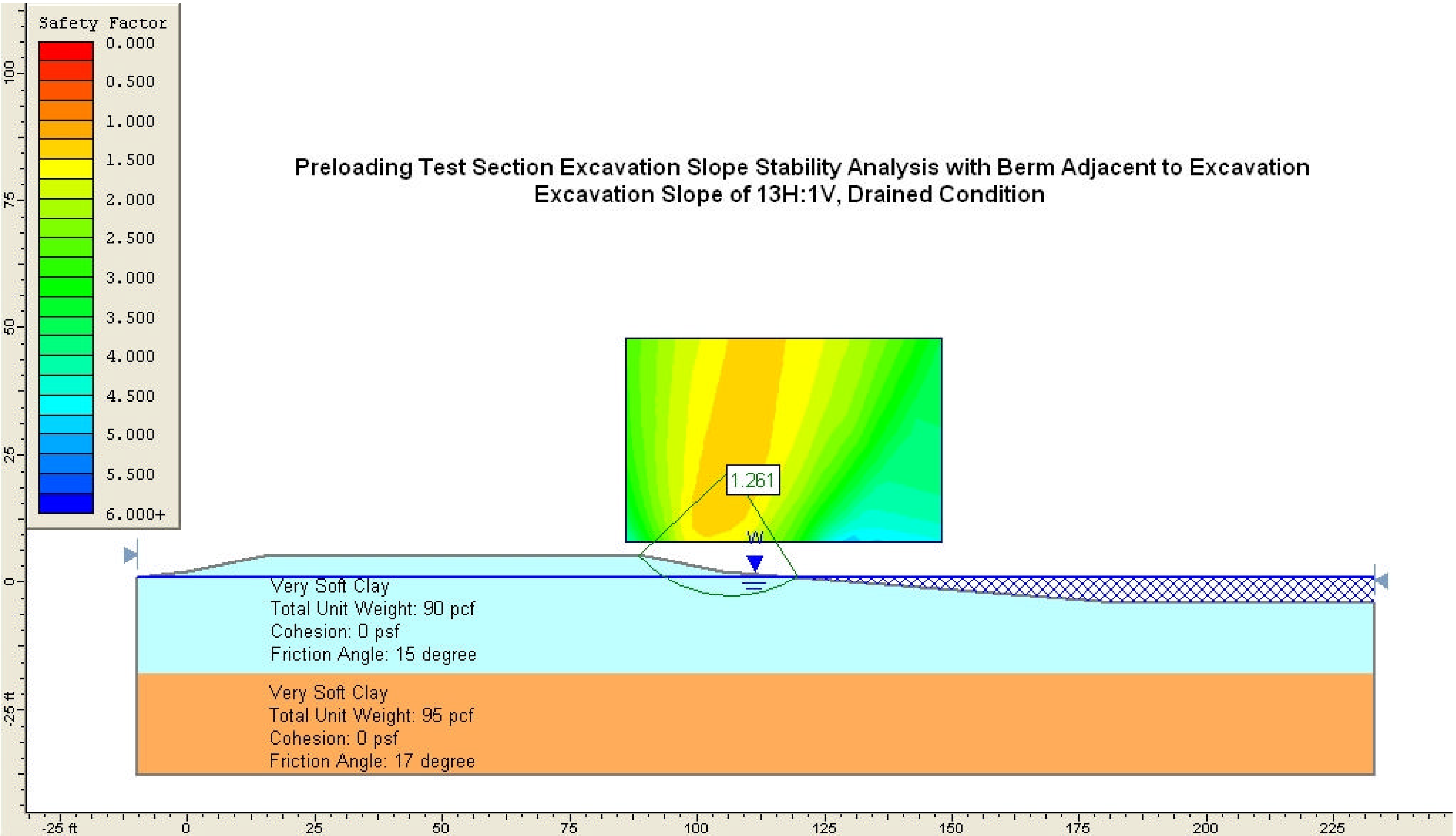


RESULTS OF STABILITY ANALYSIS

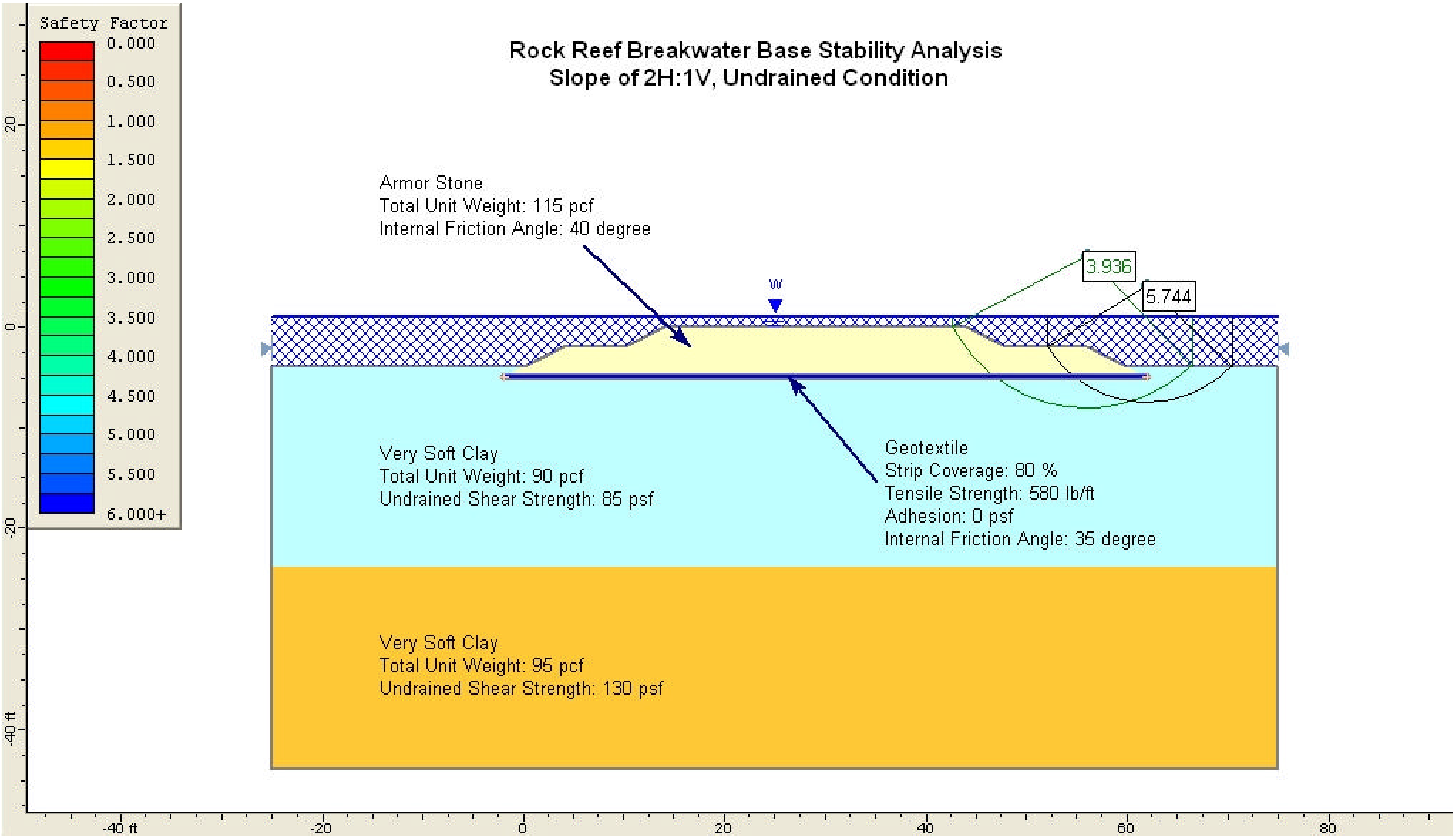
TEST SECTIONS – ROCKEFELLER REFUGE

GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

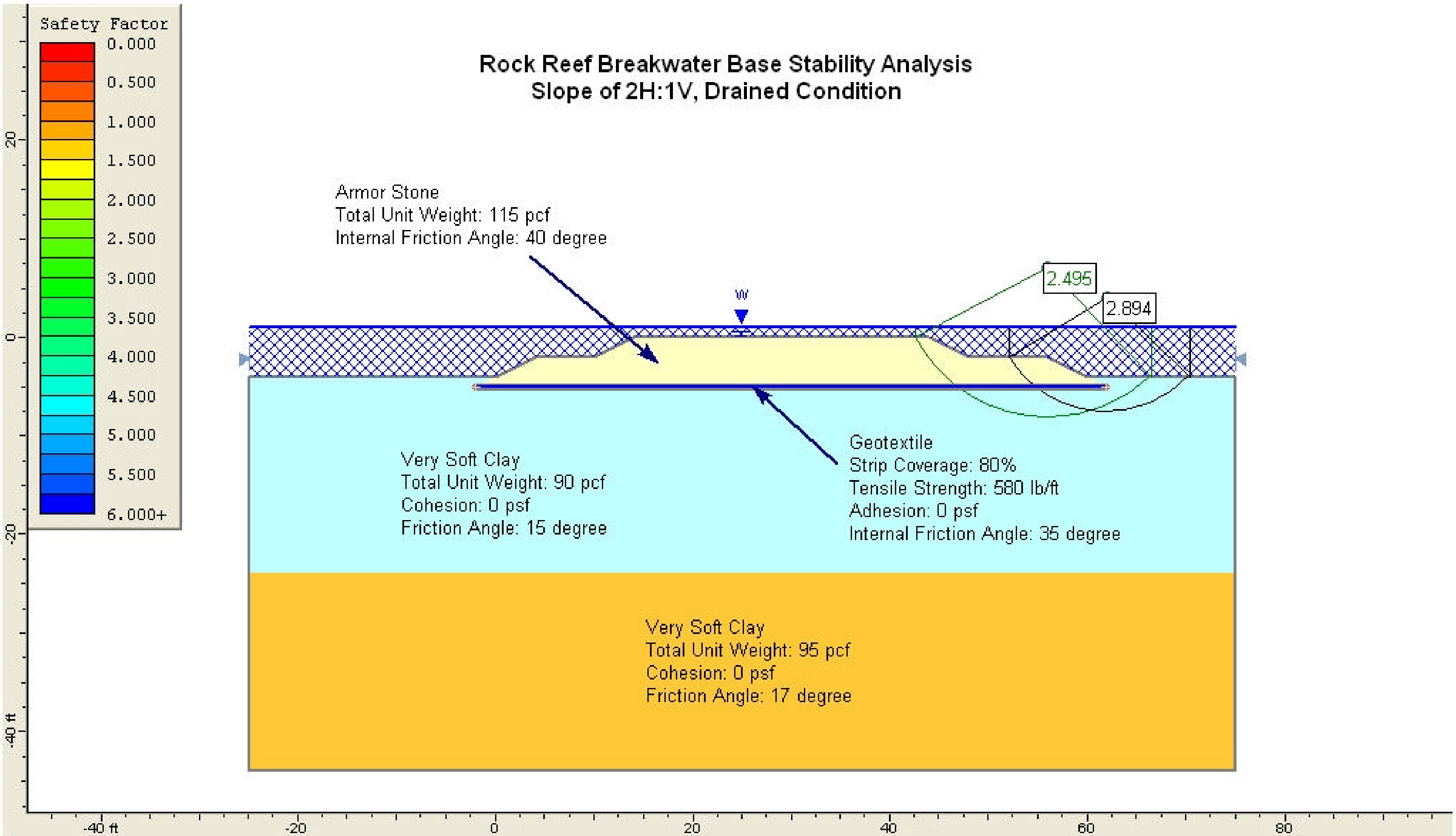


RESULTS OF STABILITY ANALYSIS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

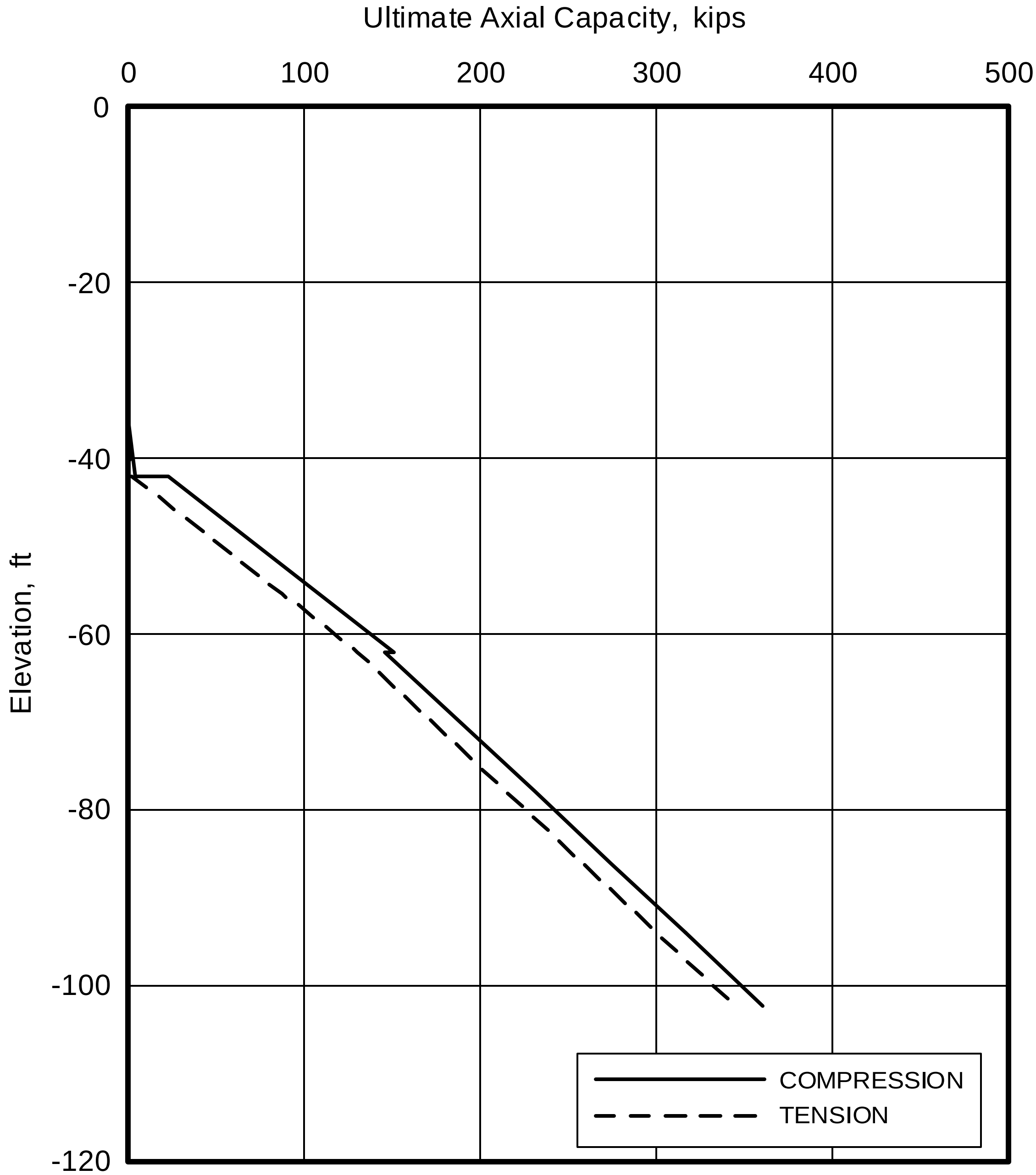


RESULTS OF STABILITY ANALYSIS

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



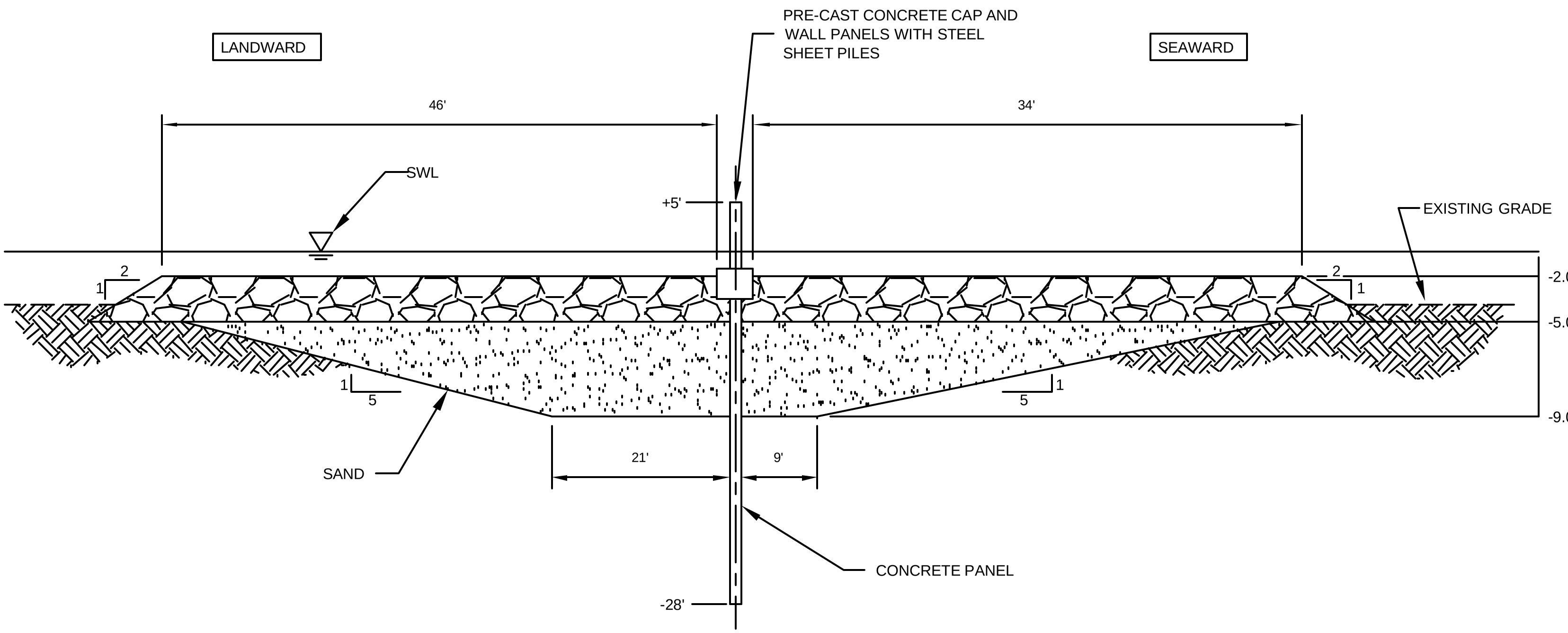
RESULTS OF STABILITY ANALYSIS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



NOTES:
1. These curves represent ultimate values for compression and tension. A safety factor of 2.0 should be applied for sustained compressive loads or transient tensile loads and a safety factor of 3.0 should be applied for sustained tensile loads.
2. These curves are for a single isolated pile. Group effects are discussed in the text.
3. The curves have been adjusted to account for negative skin friction as discussed in the text.

ULTIMATE PILE CAPACITY

16-INCH SQUARE CONCRETE PILE
TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



LEGEND

SWL STILL WATER LEVEL

ARMOR STONE

CLEAN SAND (WITH LESS THAN 30 PERCENT FINES)

RECOMMENDED SAND LATERAL EXTENT AND EXCAVATION SLOPES FOR PROPOSED CONSTRUCTION SEQUENCE
CONCRETE PANEL BREAKWATER
TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA
(NOT TO SCALE)

APPENDIX A

FUGRO

LOCATION: See Plate 2

COORDINATES: N 29°38'3.4" W 92°46'23.1"

SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					CLAY, very soft to firm, gray - with organic material to 16'				139	128	43	85
							35		141			
							36		136			
									101	110	33	77
							42		108			
									109			
									91	106	36	70

Depth scale: 5, 10, 15, 20, 25, 30, 35

CLASSIFICATION

SHEAR STRENGTH

□ Penetrometer ◇ Torvane △ Field Vane

Unconfined ▼ Triaxial ● Miniature Vane ▲

KIPS PER SQ FT

0.5 1.0 1.5 2.0 2.5

NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004

TOTAL DEPTH: 45'

CAVED DEPTH: Not Applicable

DRY AUGER: Not Applicable

WET ROTARY: 0 to 45'

BACKFILL: Cement-Bentonite Grout

LOGGER: J. Phipps

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LOG OF BORING NO. TS-1

TEST SECTIONS – ROCKEFELLER REFUGE

GULF SHORELINE STABILIZATION PROJECT

CAMERON PARISH, LOUISIANA

LOCATION: See Plate 2
COORDINATES: N 29°38'3.4" W 92°46'23.1"
SURFACE EL.: Not Available

DEPTH, FT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	WATER CONTENT, %
	CLAY, very soft to firm, gray		88
45		45.0	

SHEAR STRENGTH (KIPS PER SQ FT): □ Penetrometer; ◇ Torvane; △ Field Vane; ▼ Unconfined; ● Triaxial; ▲ Miniature Vane. Scale: 0.5, 1.0, 1.5, 2.0, 2.5

Other column headings: WATER LEVEL, SYMBOL, SAMPLES, BLOWS PER FOOT, CLASSIFICATION: UNIT DRY WT, PCF; PASSING NO. 200 SIEVE, %; LIQUID LIMIT; PLASTIC LIMIT; PLASTICITY INDEX (PI)

Depth scale: 45, 50, 55, 60, 65, 70, 75

NOTES:
1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

LOG OF BORING NO. TS-1
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29º38'4.8" W 92º46'27.7" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
0					CLAY, soft to very soft, gray - with organic material to 16'								
5									139	145	39	106	
8							36		133				
15									128	136	41	95	
20							42		114				
29									122	131	37	94	
35							46		99				

Shear strength legend: □ Penetrometer; ◇ Torvane; △ Field Vane; Unconfined ▼; Triaxial ⬢; Miniature Vane ▲

KIPS PER SQ FT: 0.5, 1.0, 1.5, 2.0, 2.5

NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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LOG OF BORING NO. TS-2

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29º38'4.8" W 92º46'27.7" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
					CLAY, soft to very soft, gray - stiff, brown and gray below 43'	45.0			28				Penetrometer ~1.2

SHEAR STRENGTH legend: □ Penetrometer, ◇ Torvane, △ Field Vane, Unconfined ▼, Triaxial ⬢, Miniature Vane ▲; KIPS PER SQ FT: 0.5, 1.0, 1.5, 2.0, 2.5

Depth scale: 45, 50, 55, 60, 65, 70, 75

NOTES:
1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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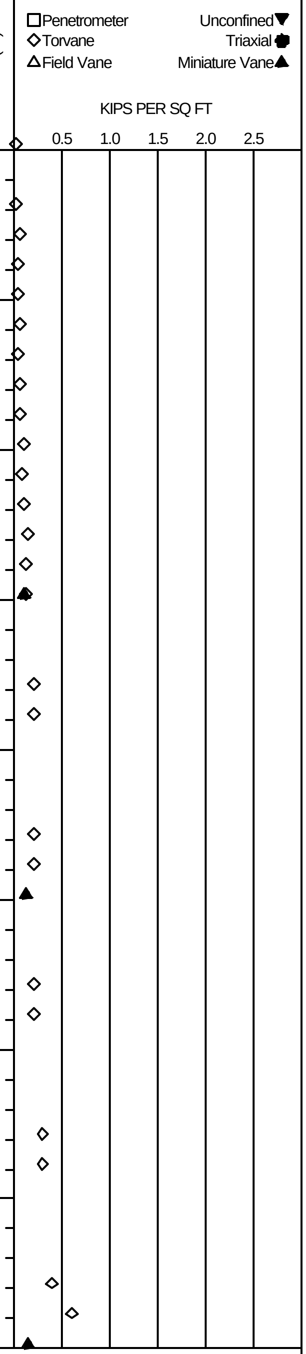
LOG OF BORING NO. TS-2
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29°38'7.2" W 92°46'34.4" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					CLAY, gray, very soft to firm - with organic material to 16'				131	127	35	92
5												
									128			
10												
15							40		121			
20												
									116	121	39	82
25							42		108			
30												
					- with silt pockets, 33' to 40'				89	98	35	63
35												
							43		106			

SHEAR STRENGTH



NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

LOG OF BORING NO. TS-3
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

LOCATION: See Plate 2
COORDINATES: N 29º38'7.2" W 92º46'34.4"
SURFACE EL.: Not Available

DEPTH, FT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
	CLAY, gray, very soft to firm	41.5						
	SANDY CLAY, soft to firm, gray	44.0			57			
45	CLAY, firm, tan	45.0						

Depth scale: 45, 50, 55, 60, 65, 70, 75

SHEAR STRENGTH, KIPS PER SQ FT: 0.5, 1.0, 1.5, 2.0, 2.5
□ Penetrometer; ◇ Torvane; △ Field Vane; Unconfined ▼; Triaxial ●; Miniature Vane ▲

NOTES:
1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

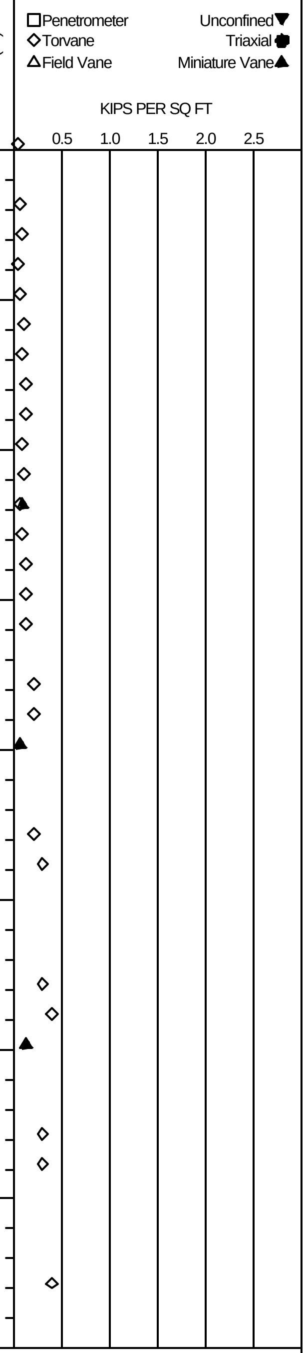
LOG OF BORING NO. TS-3
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29°38'11" W 92°46'38.3" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT
0–39					CLAY, very soft to soft, gray - with organic material to 16'	

CLASSIFICATION

UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
		131			
		122	131	39	92
		124	129	38	91
37		127			
49		107			
50		87			

SHEAR STRENGTH



NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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LOG OF BORING NO. TS-4

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29º38'11" W 92º46'38.3" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
					CLAY, very soft to soft, gray								
					- firm, brown and gray below 42'								□
					- with ferrous nodules, 43' to 45'				34	90	24	66	□
45						45.0							

Shear strength legend: □ Penetrometer; ◇ Torvane; △ Field Vane; Unconfined ▼; Triaxial ●; Miniature Vane ▲. Scale: 0.5, 1.0, 1.5, 2.0, 2.5

Depth scale: 45, 50, 55, 60, 65, 70, 75

NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

LOG OF BORING NO. TS-4

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

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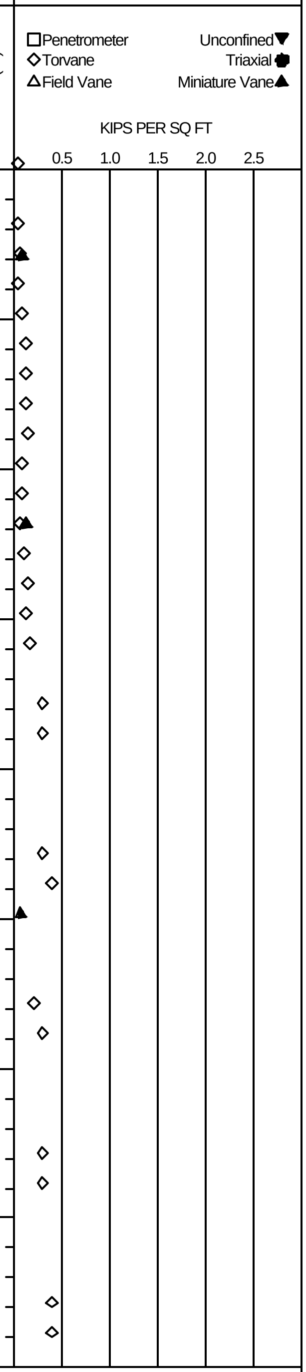
LOCATION: See Plate 2
COORDINATES: N 29º38'9.6" W 92º46'41.2"
SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					CLAY, very soft to soft, gray - with organic material to 16'				141	124	35	89
							36		136			
5												
10												
							38		131			
15												
									106	104	34	70
									85			
20												
25							44		103			
									110	116	37	79
30												
35												
									98			

SHEAR STRENGTH



NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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LOG OF BORING NO. TS-5

TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

FUGRO

LOCATION: See Plate 2
COORDINATES: N 29º38'9.6" W 92º46'41.2"
SURFACE EL.: Not Available

DEPTH, FT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	WATER CONTENT, %
	CLAY, very soft to soft, gray	41.0	
	SANDY CLAY, soft, gray - with shell fragments at 43'	44.0	90
45	CLAY, stiff, brown - with shell fragments at 44.5'	45.0	

Column headings: DEPTH, FT; WATER LEVEL; SYMBOL; SAMPLES; BLOWS PER FOOT; STRATUM DESCRIPTION; STRATUM DEPTH, FT; CLASSIFICATION: UNIT DRY WT, PCF; PASSING NO. 200 SIEVE, %; WATER CONTENT, %; LIQUID LIMIT; PLASTIC LIMIT; PLASTICITY INDEX (PI); SHEAR STRENGTH: Penetrometer, Torvane, Field Vane, Unconfined, Triaxial, Miniature Vane; KIPS PER SQ FT 0.5 1.0 1.5 2.0 2.5

Depth scale: 45, 50, 55, 60, 65, 70, 75

NOTES:
1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 18, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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LOG OF BORING NO. TS-5
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

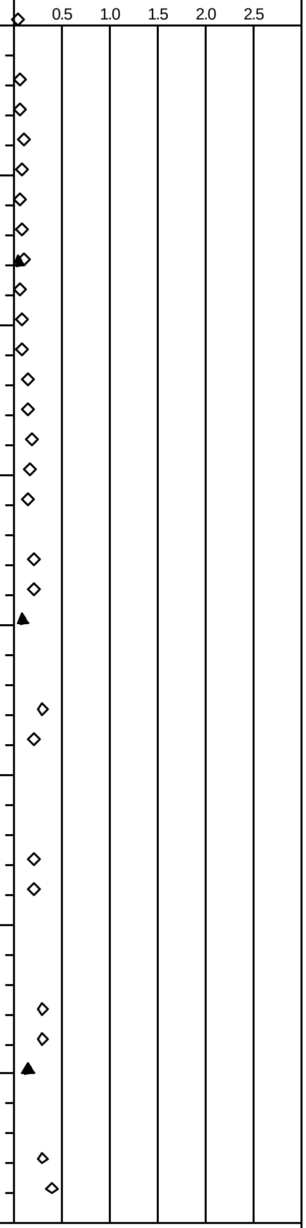
LOCATION: See Plate 2
COORDINATES: N 29°38'12.5" W 92°46'44.5"
SURFACE EL.: Not Available

STRATUM DESCRIPTION

CLAY, very soft to soft, gray
- with organic material to 16'
- with silt pockets, 33' to 40'

DEPTH, FT (approx.)	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
5.5			122	120	32	88
7.5	39		122			
19.5	42		112			
23.5			118	110	35	75
33.5			99	109	32	77
34.5	45		100			

SHEAR STRENGTH



NOTES:
1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 17, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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LOG OF BORING NO. TS-6
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29º38'12.5" W 92º46'44.5" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT) □Penetrometer ◇Torvane △Field Vane Unconfined▼ Triaxial ⬢ Miniature Vane▲
					CLAY, very soft to soft, gray	41.0							
					SANDY CLAY, firm to stiff, brown and gray - slickensided, 43' to 45'				29				□ □
45						45.0							

Depth scale: 45, 50, 55, 60, 65, 70, 75. Shear strength scale: 0.5, 1.0, 1.5, 2.0, 2.5.

NOTES:
1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 17, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

LOG OF BORING NO. TS-6
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

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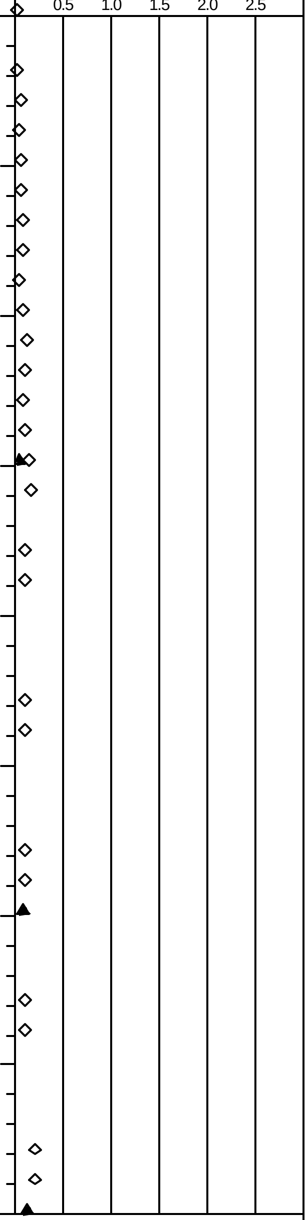
DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29º38'15.6" W 92º46'48" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
					CLAY, very soft, gray - with organic material below 0.5'							
									109			
5									177	182	55	127
10												
15							45		102			
									121	110	35	75
20												
25									113	120	43	77
30							42		107			
35					- with silt pockets, 38' to 40'							
							47		89			

SHEAR STRENGTH

□Penetrometer ◇Torvane △Field Vane Unconfined▼ Triaxial● Miniature Vane▲

KIPS PER SQ FT: 0.5 1.0 1.5 2.0 2.5



NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 17, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

R:\LAKECH~1\0604\0604-1~1\DRAFTING\0604-1370.GPJ FUGRO_SO (LAB DATA) 8/16/2004

LOG OF BORING NO. TS-7
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

LOCATION: See Plate 2
COORDINATES: N 29°38'15.6" W 92°46'48"
SURFACE EL.: Not Available

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
40–45					CLAY, firm, gray - with silt pockets, 40' to 45' - with shell fragments, 43' to 45'	45.0			82				

SHEAR STRENGTH legend: Penetrometer, Torvane, Field Vane, Unconfined, Triaxial, Miniature Vane. KIPS PER SQ FT: 0.5, 1.0, 1.5, 2.0, 2.5

Depth scale: 45, 50, 55, 60, 65, 70, 75

NOTES:
1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 17, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

LOG OF BORING NO. TS-7
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

LOCATION: See Plate 2
COORDINATES: N 29º38'7.2" W 92º46'31.4"
SURFACE EL.: Not Available

Columns: DEPTH, FT | WATER LEVEL | SYMBOL | SAMPLES | BLOWS PER FOOT | STRATUM DESCRIPTION | STRATUM DEPTH, FT | CLASSIFICATION | SHEAR STRENGTH

DEPTH, FT: 5, 10, 15, 20, 25, 30, 35

STRATUM DESCRIPTION

CLAY, very soft to soft, gray
- with organic material below 0.5'

CLASSIFICATION

UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
38		126			
		122	126	41	85
		130			
		106			
		105	111	32	79
43		110			
		96	101	35	66

SHEAR STRENGTH

□Penetrometer ◇Torvane △Field Vane
Unconfined▼ Triaxial⬢ Miniature Vane▲

KIPS PER SQ FT

0.5 1.0 1.5 2.0 2.5

NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 19, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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LOG OF BORING NO. TS-8
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29º38'7.2" W 92º46'31.4" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
					CLAY, very soft to soft, gray	41.0							
					SANDY CLAY, firm, gray				76				
45						45.0							
50													
55													
60													
65													
70													
75													

Shear strength legend: □ Penetrometer; ◇ Torvane; △ Field Vane; Unconfined ▼; Triaxial ⬢; Miniature Vane ▲. Scale: 0.5, 1.0, 1.5, 2.0, 2.5

NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 19, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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LOG OF BORING NO. TS-8
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

FUGRO

LOCATION: See Plate 2
COORDINATES: N 29°38'14.2" W 92°46'46.2"
SURFACE EL.: Not Available

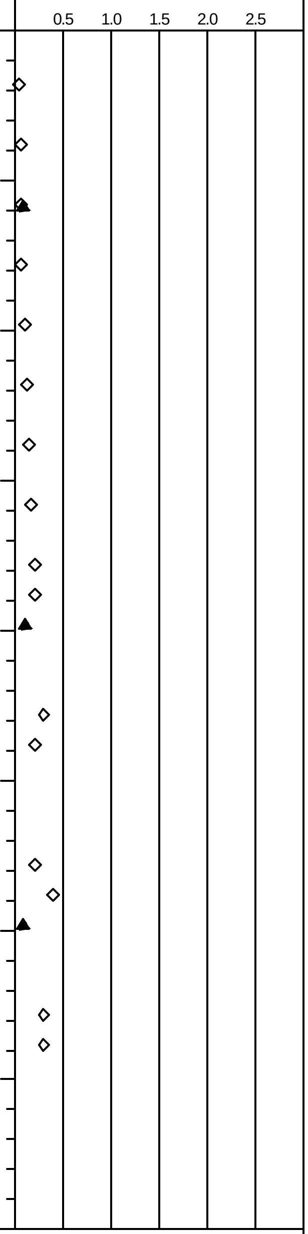
STRATUM DESCRIPTION	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)
CLAY, very soft to soft, gray - with organic material to 16'						
	40		121			
			108	121	38	83
			121			
	41		116			
			105	110	35	75
	41		103			
			103	114	38	76

DEPTH, FT: 5, 10, 15, 20, 25, 30, 35

SHEAR STRENGTH

□Penetrometer ◇Torvane △Field Vane
Unconfined▼ Triaxial⬢ Miniature Vane▲

KIPS PER SQ FT



NOTES:

1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 19, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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LOG OF BORING NO. TS-9
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

DEPTH, FT	WATER LEVEL	SYMBOL	SAMPLES	BLOWS PER FOOT	LOCATION: See Plate 2 COORDINATES: N 29º38'14.2" W 92º46'46.2" SURFACE EL.: Not Available STRATUM DESCRIPTION	STRATUM DEPTH, FT	UNIT DRY WT, PCF	PASSING NO. 200 SIEVE, %	WATER CONTENT, %	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX (PI)	SHEAR STRENGTH (KIPS PER SQ FT)
					CLAY, very soft to soft, gray	41.0							
					SANDY CLAY, firm to stiff, brown and gray				28				
45						45.0							
50													
55													
60													
65													
70													
75													

Shear strength legend: □ Penetrometer, ◇ Torvane, △ Field Vane, Unconfined ▼, Triaxial ●, Miniature Vane ▲. Scale: 0.5, 1.0, 1.5, 2.0, 2.5

NOTES:
1. Water level was not measured during drilling as 5 ft of standing water was encountered at the boring locations.
2. Terms and symbols defined on Plate A-10.

DATE: June 19, 2004
TOTAL DEPTH: 45'
CAVED DEPTH: Not Applicable
DRY AUGER: Not Applicable
WET ROTARY: 0 to 45'
BACKFILL: Cement-Bentonite Grout
LOGGER: J. Phipps

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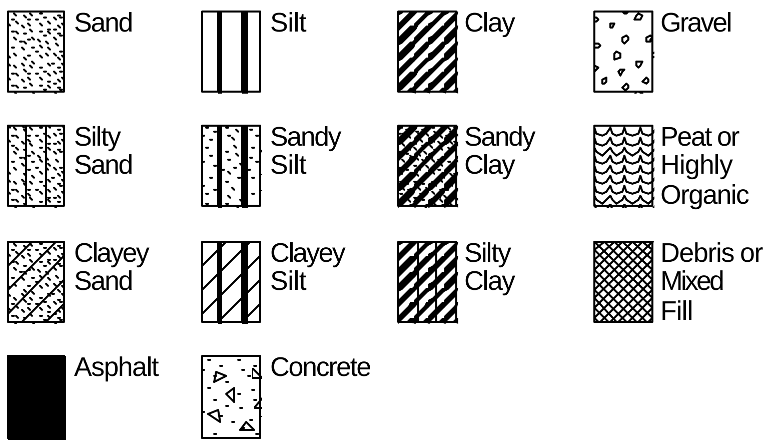
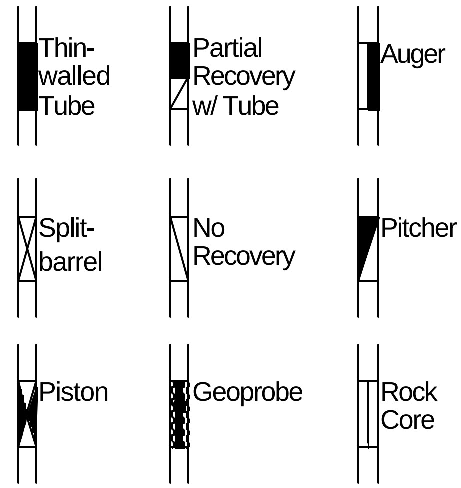
LOG OF BORING NO. TS-9
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

SOIL TYPES



SAMPLER TYPES



SOIL GRAIN SIZE

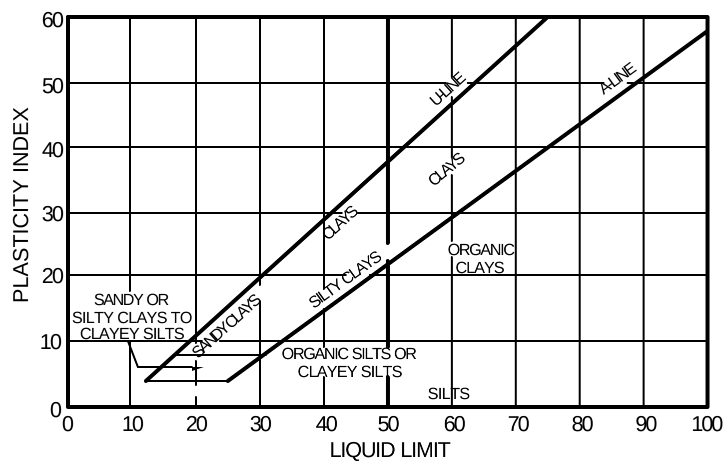
U.S. Standard Sieve

6" 3" 3/4" 4 10 40 200

Boulders	Cobbles	Gravel		Sand			Silt	Clay
		Coarse	Fine	Coarse	Medium	Fine		

152 76.2 19.1 4.76 2.00 0.420 0.074 0.002 (mm)

PLASTICITY CHART



SOIL STRUCTURE

Slickensided .... Having planes of weakness that appear slick and glossy.
Fissured .... Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
Pocket .... Inclusion of material of different texture that is smaller than the diameter of the sample.
Parting .... Inclusion less than 1/8 inch thick extending through the sample.
Seam .... Inclusion 1/8 inch to 3 inches thick extending through the sample.
Layer .... Inclusion greater than 3 inches thick extending through the sample.
Laminated .... Soil sample composed of alternating partings or seams of different soil type.
Interlayered .... Soil sample composed of alternating layers of different soil type.
Intermixed .... Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
Calcareous .... Having appreciable quantities of carbonate.
Carbonate .... Having more than 50% carbonate content.

TERMS AND SYMBOLS USED ON BORING LOGS

SOIL CLASSIFICATION (1 of 2)

STANDARD PENETRATION TEST (SPT)

A 2-in.-OD, 1-3/8-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

SPLIT-BARREL SAMPLER DRIVING RECORD

Blows Per Foot	Description
25	25 blows drove sampler 12 inches, after initial 6 inches of seating.
50/7"	50 blows drove sampler 7 inches, after initial 6 inches of seating.
Ref/3"	50 blows drove sampler 3 inches during initial 6-inch seating interval.

NOTE: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

DENSITY OF GRANULAR SOILS

Descriptive Term	*Relative Density, %	**Blows Per Foot (SPT)
Very Loose	< 15	0 to 4
Loose	15 to 35	5 to 10
Medium Dense	35 to 65	11 to 30
Dense	65 to 85	31 to 50
Very Dense	> 85	> 50

*Estimated from sampler driving record.
**Requires correction for depth, groundwater level, and grain size.

STRENGTH OF COHESIVE SOILS

Term	Undrained Shear Strength, ksf	Blows Per Foot (SPT) (approximate)
Very Soft	< 0.25	0 to 2
Soft	0.25 to 0.50	2 to 4
Firm	0.50 to 1.00	4 to 8
Stiff	1.00 to 2.00	8 to 16
Very Stiff	2.00 to 4.00	16 to 32
Hard	> 4.00	> 32

SHEAR STRENGTH TEST METHOD

U - Unconfined Q = Unconsolidated - Undrained Triaxial

P = Pocket Penetrometer T = Torvane V = Miniature Vane F = Field Vane

HAND PENETROMETER CORRECTION

Our experience has shown that the hand penetrometer generally overestimates the in-situ undrained shear strength of over consolidated Pleistocene Gulf Coast clays. These strengths are partially controlled by the presence of macroscopic soil defects such as slickensides, which generally do not influence smaller scale tests like the hand penetrometer. Based on our experience, we have adjusted these field estimates of the undrained shear strength of natural, overconsolidated Pleistocene Gulf Coast soils by multiplying the measured penetrometer reading by a factor of 0.6. These adjusted strength estimates are recorded in the "Shear Strength" column on the boring logs. Except as described in the text, we have not adjusted estimates of the undrained shear strength for projects located outside of the Pleistocene Gulf Coast formations.

Information on each boring log is a compilation of subsurface conditions and soil or rock classifications obtained from the field as well as from laboratory testing of samples. Strata have been interpreted by commonly accepted procedures. The stratum lines on the logs may be transitional and approximate in nature. Water level measurements refer only to those observed at the time and places indicated, and can vary with time, geologic condition, or construction activity.

TERMS AND SYMBOLS USED ON BORING LOGS

SOIL CLASSIFICATION (2 of 2)

APPENDIX B

FIELD VANE SHEAR TEST
0604-1370
ROCKERFELLER REFUGE

Report No. 0604-1370

BORING NUMBER **VS-1**

COORDINATES 29° 38' 4"
92° 46' 23"

DATE 06/18/04

Vane Type: **Rectangular Vane**

Housing Used: **Yes / No**

FRICTION TEST		SIZE	DEPTH		TORVANE				REMOLDED			
Torque (T) (in.lbs)	Time To Failure	VANE (D)	Borehole (5 x dia. ft)	Vane Tip (feet)	Torque (T) (in.lbs)	Torque (T) (ft.lbs)	Time To Failure	Shear Strength (s) (lbf/ft^2)	Torque (T) (in.lbs)	Torque (T) (ft.lbs)	Time To Failure	Shear Strength (s)
10	1:48	3.63	**0.8**	**2.5**	160	13.3	1:32	133.3	35	2.9	54 sec	29.2
			3.3	**5**	170	14.2	1:26	141.7	40	3.3	1:11	33.3
			5.8	**7.5**	180	15.0	1:48	150.0	45	3.8	1:06	37.5
			8.3	**10**	150	12.5	1:39	125.0	30	2.5	51 sec	25.0
			10.8	**12.5**	160	13.3	1:57	133.3	35	2.9	1:08	29.2
			13.3	**15**	185	15.4	2:04	154.2	65	5.4	1:27	54.2
			15.8	**17.5**	200	16.7	1:59	166.7	70	5.8	1:22	58.3
			18.3	**20**	225	18.8	2:13	187.5	75	6.3	1:25	62.5

Crew:

For: Vane Dia = 1.5" k = 141.1 (1/ft^3)
Vane Dia = 2.0" k = 59.5 (1/ft^3)
Vane Dia = 2.5" k = 30.5 (1/ft^3)
Vane Dia =3.63" k = 10.0 (1/ft^3)

Shear Strength (psf): **s = T x k,** where **k = 1 / K**, and **K = 0.0021(D^3)**
(The vane Dia. (D) is entered in inches and the formula converts to ft^3)
(Torque in (ft-lbs) is used in the Shear Strength formula: s = T x k)
(Torque in (in-lbs) is divided by 12 to convert to the (ft-lbs) column)

PLATE B-1

FIELD VANE SHEAR TEST
0604-1370
ROCKERFELLER REFUGE

BORING NUMBER **VS-2**

COORDINATES 29° 38' 10"
92° 46' 28"

DATE 06/18/04

Vane Type: **Rectangular Vane**

Housing Used: **Yes / No**

FRICTION TEST		SIZE	DEPTH		TORVANE				REMOLDED			
Torque (T) (in.lbs)	Time To Failure	VANE (D)	Borehole (5 x dia. ft)	Vane Tip (feet)	Torque (T) (in.lbs)	Torque (T) (ft.lbs)	Time To Failure	Shear Strength (s) (lbf/ft^2)	Torque (T) (in.lbs)	Torque (T) (ft.lbs)	Time To Failure	Shear Strength (s)
		3.63	**0.8**	**2.5**	150	12.5	2:00	125.0	35	2.9	1:30	29.2
15	1:17		**3.3**	**5**	160	13.3	1:48	133.3	40	3.3	1:25	33.3
			5.8	**7.5**	120	10.0	1:55	100.0	30	2.5	1:48	25.0
			8.3	**10**	135	11.3	2:10	112.5	35	2.9	2:00	29.2
			10.8	**12.5**	175	14.6	2:05	145.8	55	4.6	1:52	45.8
			13.3	**15**	200	16.7	1:50	166.7	65	5.4	1:43	54.2
			15.8	**17.5**	220	18.3	1:54	183.3	70	5.8	1:37	58.3
			18.3	**20**	220	18.3	2:07	183.3	75	6.3	1:50	62.5

Crew:

For: Vane Dia = 1.5" k = 141.1 (1/ft^3)
Vane Dia = 2.0" k = 59.5 (1/ft^3)
Vane Dia = 2.5" k = 30.5 (1/ft^3)
Vane Dia =3.63" k = 10.0 (1/ft^3)

Shear Strength (psf): **s = T x k,** where **k = 1 / K**, and **K = 0.0021(D^3)**
(The vane Dia. (D) is entered in inches and the formula converts to ft^3)
(Torque in (ft-lbs) is used in the Shear Strength formula: s = T x k)
(Torque in (in-lbs) is divided by 12 to convert to the (ft-lbs) column)

FIELD VANE SHEAR TEST
0604-1370
ROCKERFELLER REFUGE

BORING NUMBER **VS-3**
COORDINATES 29° 38' 10.7"
92° 46' 34.5"
DATE 06/18/04

Vane Type: **Rectangular Vane**

Housing Used: **Yes / No**

FRICTION TEST		SIZE	DEPTH		TORVANE				REMOLDED			
Torque (T) (in.lbs)	**Time To Failure**	**VANE (D)**	**Borehole (5 x dia. ft)**	**Vane Tip (feet)**	**Torque (T) (in.lbs)**	**Torque (T) (ft.lbs)**	**Time To Failure**	**Shear Strength (s) (lbf/ft^2)**	**Torque (T) (in.lbs)**	**Torque (T) (ft.lbs)**	**Time To Failure**	**Shear Strength (s)**
		3.63	**0.8**	**2.5**	145	12.1	1:30	120.8	30	2.5	1:20	25.0
15	52 sec.		**3.3**	**5**	165	13.8	1:48	137.5	45	3.8	1:37	37.5
			5.8	**7.5**	150	12.5	1:52	125.0	30	2.5	1:22	25.0
			8.3	**10**	155	12.9	1:43	129.2	50	4.2	1:34	41.7
			10.8	**12.5**	190	15.8	1:37	158.3	60	5.0	1:50	50.0
			13.3	**15**	210	17.5	1:55	175.0	70	5.8	1:23	58.3
			15.8	**17.5**	220	18.3	1:38	183.3	80	6.7	1:55	66.7
			18.3	**20**	240	20.0	2:07	200.0	95	7.9	2:10	79.2

Crew:

Shear Strength (psf): **s = T x k,** where **k = 1 / K**, and **K = 0.0021(D^3)**
(The vane Dia. (D) is entered in inches and the formula converts to ft^3)
(Torque in (ft-lbs) is used in the Shear Strength formula: s = T x k)
(Torque in (in-lbs) is divided by 12 to convert to the (ft-lbs) column)

For: Vane Dia = 1.5" k = 141.1 (1/ft^3)
Vane Dia = 2.0" k = 59.5 (1/ft^3)
Vane Dia = 2.5" k = 30.5 (1/ft^3)
Vane Dia =3.63" k = 10.0 (1/ft^3)

FIELD VANE SHEAR TEST
0604-1370
ROCKERFELLER REFUGE

BORING NUMBER **VS-4**
COORDINATES 29° 38' 12.9"
92° 46' 41.5"
DATE 06/17/04

Vane Type: **Rectangular Vane**

Housing Used: **Yes / No**

FRICTION TEST		SIZE	DEPTH		TORVANE				REMOLDED			
Torque (T) (in.lbs)	Time To Failure	VANE (D)	Borehole (5 x dia. ft)	Vane Tip (feet)	Torque (T) (in.lbs)	Torque (T) (ft.lbs)	Time To Failure	Shear Strength (s) (lbf/ft^2)	Torque (T) (in.lbs)	Torque (T) (ft.lbs)	Time To Failure	Shear Strength (s)
15	1:05	3.63	**0.8**	**2.5**	155	12.9	1:37	129.2	35	2.9	1:20	29.2
			3.3	**5**	120	10.0	2:05	100.0	30	2.5	1:50	25.0
			5.8	**7.5**	145	12.1	1:50	120.8	40	3.3	1:44	33.3
			8.3	**10**	155	12.9	1:43	129.2	45	3.8	1:37	37.5
			10.8	**12.5**	185	15.4	2:00	154.2	45	3.8	1:50	37.5
			13.3	**15**	210	17.5	1:50	175.0	55	4.6	2:00	45.8
			15.8	**17.5**	195	16.3	2:10	162.5	50	4.2	2:10	41.7
			18.3	**20**	200	16.7	2:20	166.7	50	4.2	1:52	41.7

Crew:

Shear Strength (psf): **s = T x k,** where **k = 1 / K**, and **K = 0.0021(D^3)**
(The vane Dia. (D) is entered in inches and the formula converts to ft^3)
(Torque in (ft-lbs) is used in the Shear Strength formula: s = T x k)
(Torque in (in-lbs) is divided by 12 to convert to the (ft-lbs) column)

For: Vane Dia = 1.5" k = 141.1 (1/ft^3)
Vane Dia = 2.0" k = 59.5 (1/ft^3)
Vane Dia = 2.5" k = 30.5 (1/ft^3)
Vane Dia =3.63" k = 10.0 (1/ft^3)

FIELD VANE SHEAR TEST
0604-1370
ROCKERFELLER REFUGE

BORING NUMBER **VS-5**

COORDINATES 29° 38' 13.8"
92° 46' 44.4"

DATE 06/17/04

Vane Type: **Rectangular Vane**

Housing Used: **Yes / No**

FRICTION TEST		SIZE	DEPTH		TORVANE				REMOLDED			
Torque (T) (in.lbs)	**Time To Failure**	**VANE (D)**	**Borehole (5 x dia. ft)**	**Vane Tip (feet)**	**Torque (T) (in.lbs)**	**Torque (T) (ft.lbs)**	**Time To Failure**	**Shear Strength (s) (lbf/ft^2)**	**Torque (T) (in.lbs)**	**Torque (T) (ft.lbs)**	**Time To Failure**	**Shear Strength (s)**
10	1:05	3.63	**0.8**	**2.5**	160	13.3	2:04	133.3	30	2.5	1:43	25.0
			3.3	**5**	180	15.0	1:50	150.0	50	4.2	1:21	41.7
			5.8	**7.5**	155	12.9	1:45	129.2	40	3.3	1:27	33.3
			8.3	**10**	150	12.5	1:52	125.0	35	2.9	1:38	29.2
			10.8	**12.5**	150	12.5	2:10	125.0	40	3.3	1:45	33.3
			13.3	**15**	175	14.6	1:48	145.8	45	3.8	1:40	37.5
			15.8	**17.5**	190	15.8	2:15	158.3	60	5.0	1:52	50.0
			18.3	**20**	215	17.9	2:00	179.2	65	5.4	1:48	54.2

Crew:

Shear Strength (psf): **s = T x k,** where **k = 1 / K**, and **K = 0.0021(D^3)**
(The vane Dia. (D) is entered in inches and the formula converts to ft^3)
(Torque in (ft-lbs) is used in the Shear Strength formula: s = T x k)
(Torque in (in-lbs) is divided by 12 to convert to the (ft-lbs) column)

For: Vane Dia = 1.5" k = 141.1 (1/ft^3)
Vane Dia = 2.0" k = 59.5 (1/ft^3)
Vane Dia = 2.5" k = 30.5 (1/ft^3)
Vane Dia =3.63" k = 10.0 (1/ft^3)

FIELD VANE SHEAR TEST
0604-1370
ROCKERFELLER REFUGE

BORING NUMBER **VS-6**
COORDINATES 29° 38' 16.1"
92° 46' 48"
DATE 06/17/04

Vane Type: **Rectangular Vane**

Housing Used: **Yes / No**

FRICTION TEST		SIZE	DEPTH		TORVANE				REMOLDED			
Torque (T) (in.lbs)	**Time To Failure**	**VANE (D)**	**Borehole (5 x dia. ft)**	**Vane Tip (feet)**	**Torque (T) (in.lbs)**	**Torque (T) (ft.lbs)**	**Time To Failure**	**Shear Strength (s) (lbf/ft^2)**	**Torque (T) (in.lbs)**	**Torque (T) (ft.lbs)**	**Time To Failure**	**Shear Strength (s)**
15	1:21	3.63	**0.8**	**2.5**	160	13.3	1:48	133.3	30	2.5	1:15	25.0
			3.3	**5**	70	5.8	2:10	58.3	20	1.7	1:30	16.7
			5.8	**7.5**	170	14.2	1:52	141.7	40	3.3	1:20	33.3
			8.3	**10**	160	13.3	1:37	133.3	40	3.3	1:10	33.3
			10.8	**12.5**	195	16.3	1:52	162.5	50	4.2	1:20	41.7
			13.3	**15**	225	18.8	2:20	187.5	55	4.6	1:48	45.8
			15.8	**17.5**	215	17.9	2:10	179.2	50	4.2	1:40	41.7
			18.3	**20**	210	17.5	2:00	175.0	60	5.0	1:30	50.0

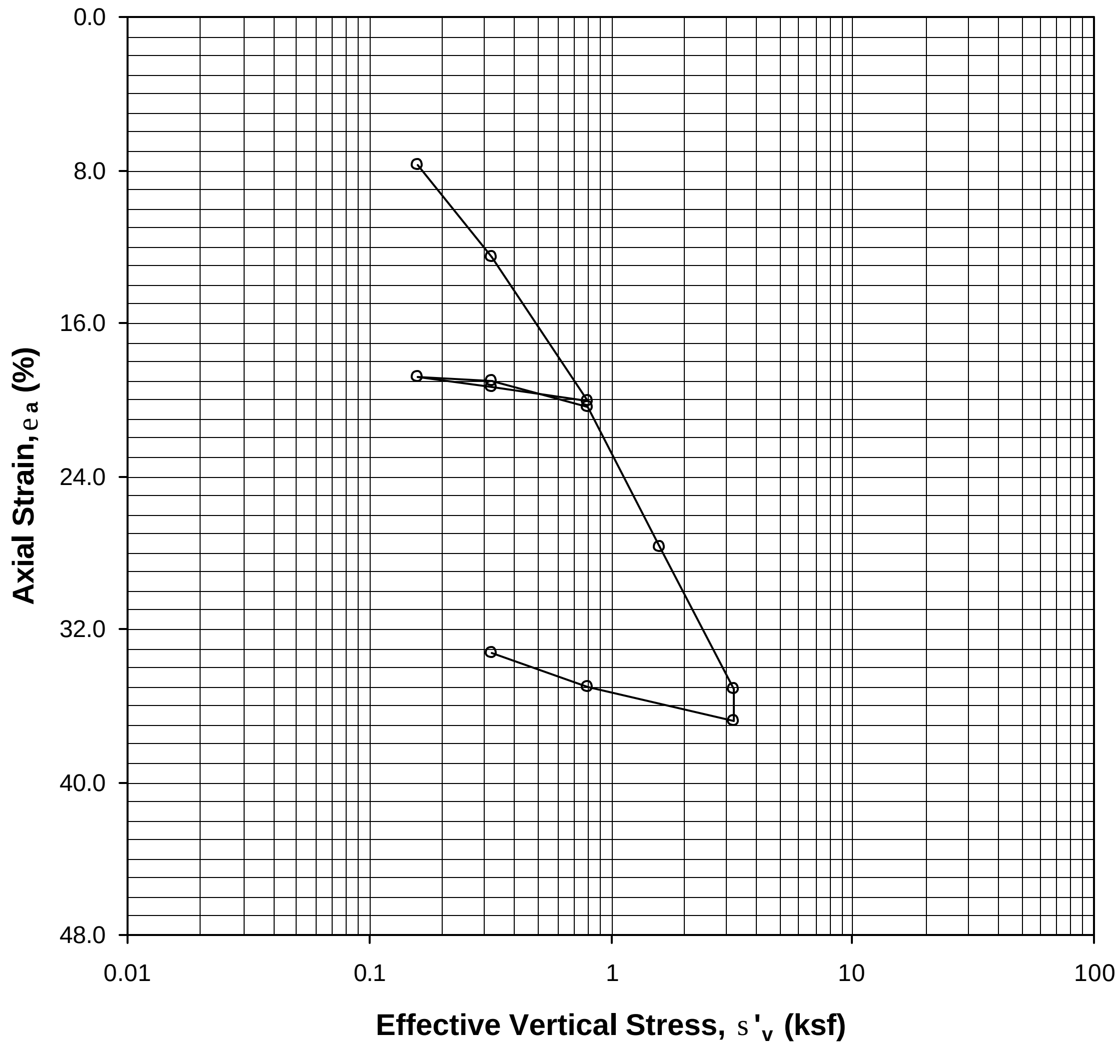
Crew:

Shear Strength (psf): **s = T x k,** where **k = 1 / K**, and **K = 0.0021(D^3)**
(The vane Dia. (D) is entered in inches and the formula converts to ft^3)
(Torque in (ft-lbs) is used in the Shear Strength formula: s = T x k)
(Torque in (in-lbs) is divided by 12 to convert to the (ft-lbs) column)

For: Vane Dia = 1.5" k = 141.1 (1/ft^3)
Vane Dia = 2.0" k = 59.5 (1/ft^3)
Vane Dia = 2.5" k = 30.5 (1/ft^3)
Vane Dia =3.63" k = 10.0 (1/ft^3)

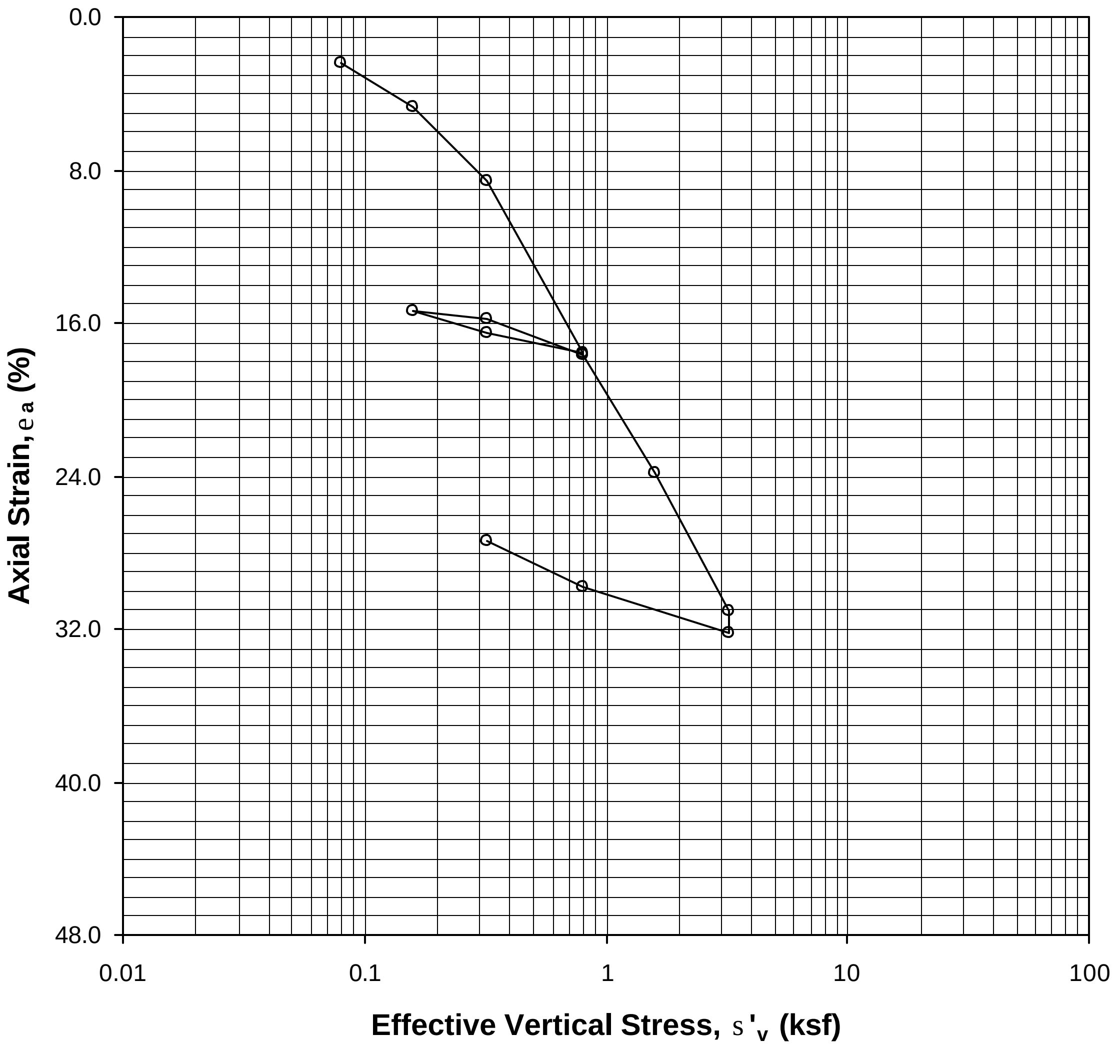
APPENDIX C

BORING: TS-5
PENETRATION: 8 ft
MATERIAL: CLAY, very soft to soft, gray
WATER CONTENT: 132.5%
INITIAL VOID RATIO: 3.90
FINAL VOID RATIO: 2.35
SPECIFIC GRAVITY: 2.7 (assumed)



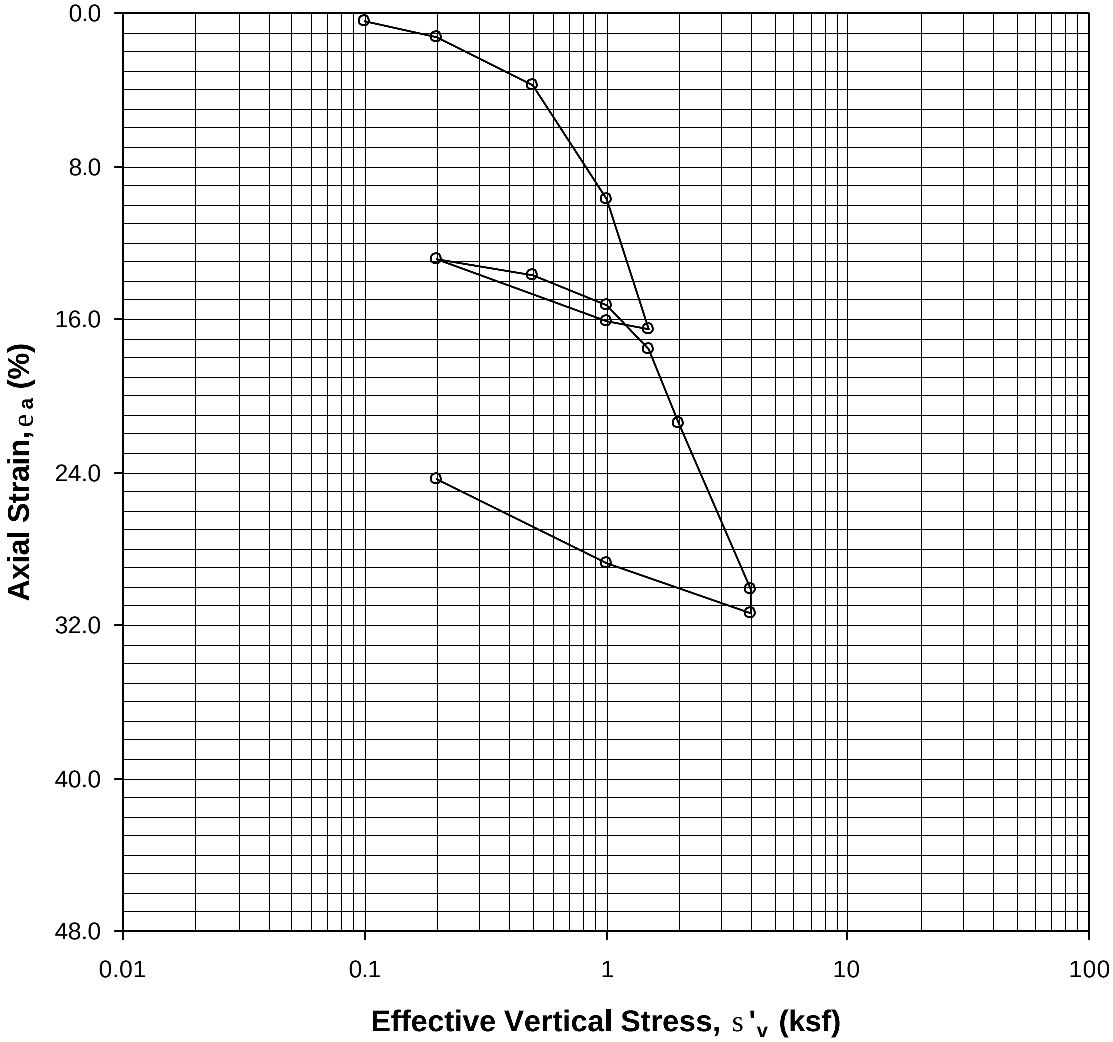
CONSOLIDATION TEST RESULTS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

BORING: TS-6
PENETRATION: 15 ft
MATERIAL: CLAY, very soft to soft, gray
WATER CONTENT: 99.5%
INITIAL VOID RATIO: 2.82
FINAL VOID RATIO: 1.94
SPECIFIC GRAVITY: 2.7 (assumed)



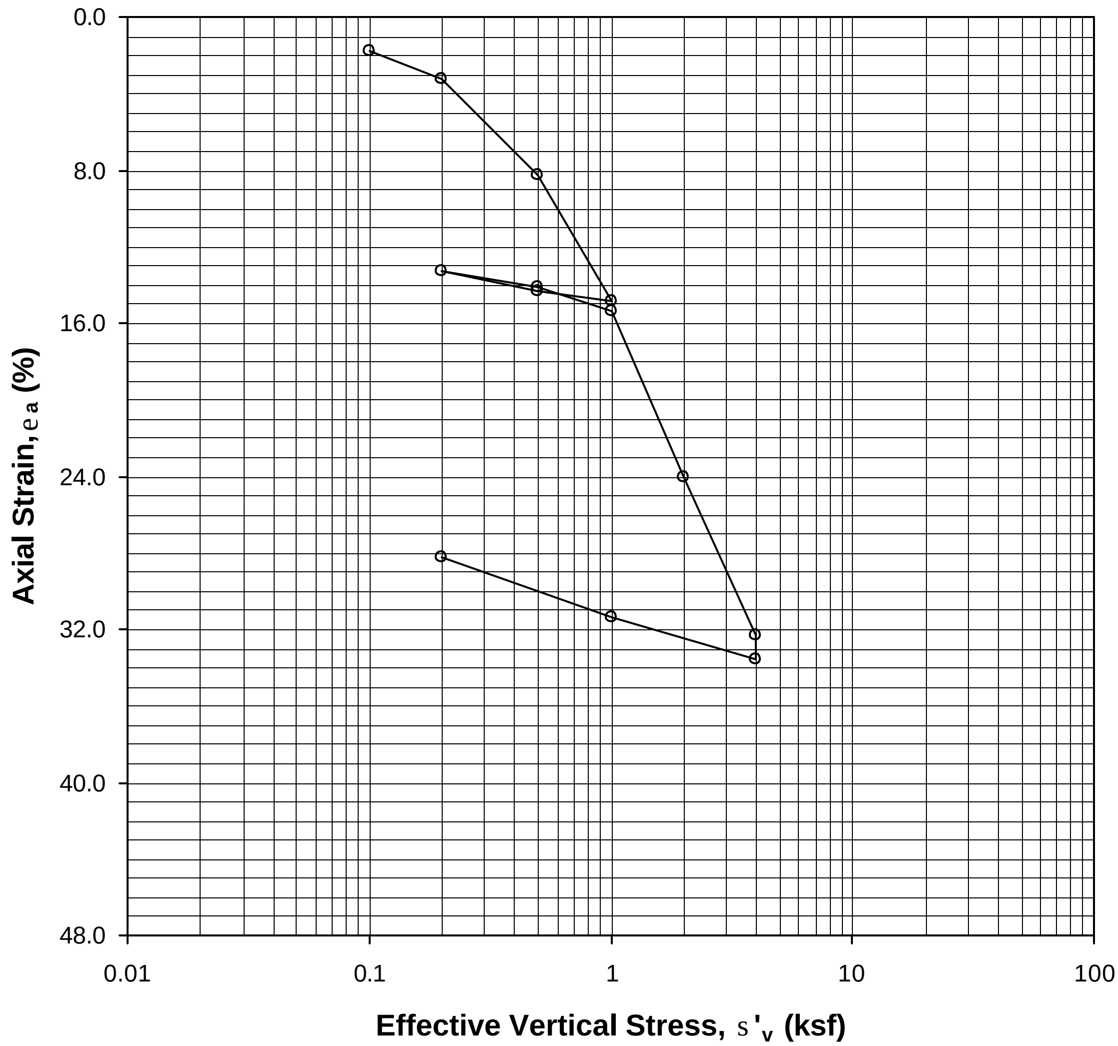
CONSOLIDATION TEST RESULTS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

BORING: TS-6
PENETRATION: 30 ft
MATERIAL: CLAY, very soft to soft, gray
WATER CONTENT: 105.9%
INITIAL VOID RATIO: 2.90
FINAL VOID RATIO: 2.09
SPECIFIC GRAVITY: 2.7 (assumed)



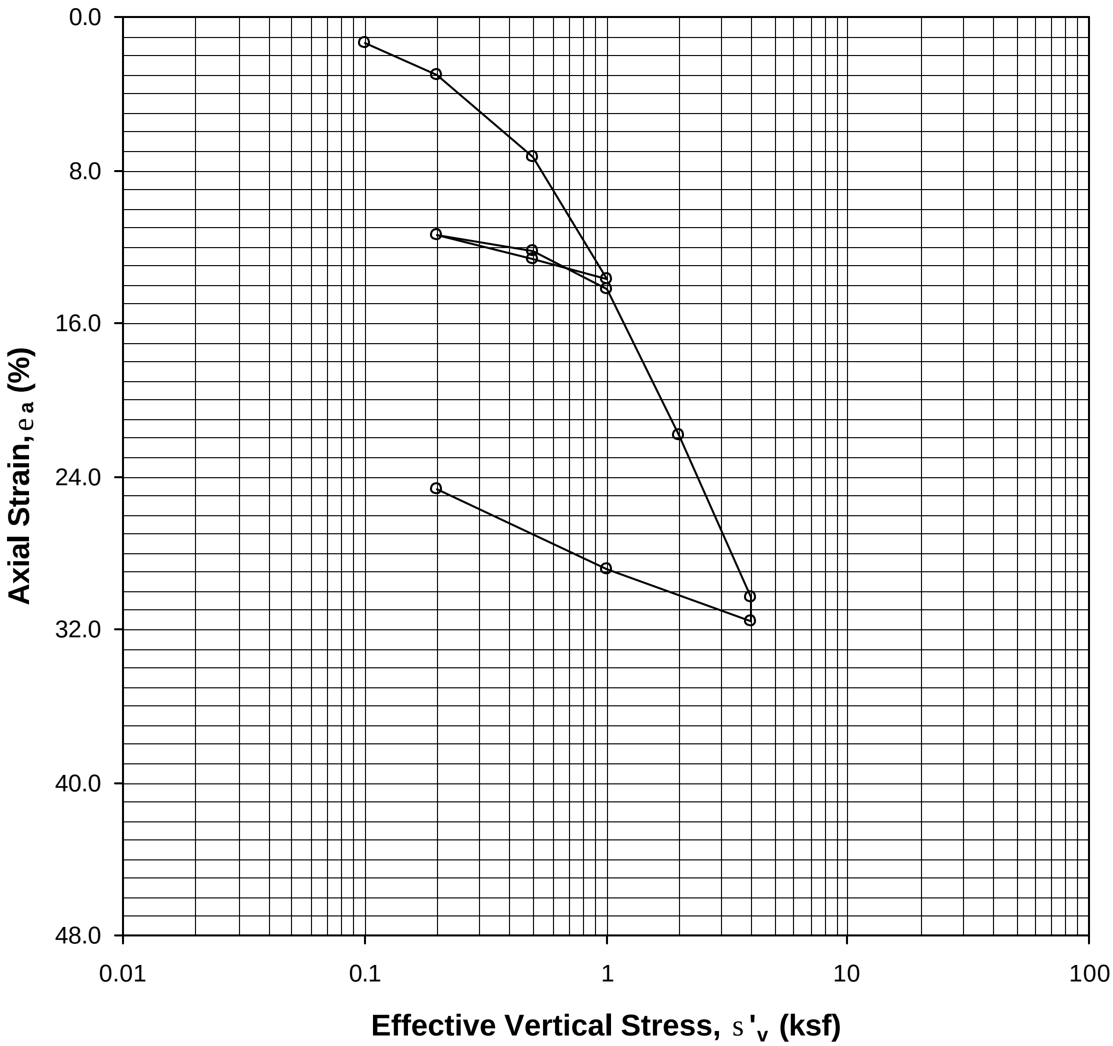
CONSOLIDATION TEST RESULTS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

BORING: TS-7
PENETRATION: 25 ft
MATERIAL: CLAY, very soft, gray
WATER CONTENT: 97.6%
INITIAL VOID RATIO: 2.66
FINAL VOID RATIO: 1.77
SPECIFIC GRAVITY: 2.7 (assumed)



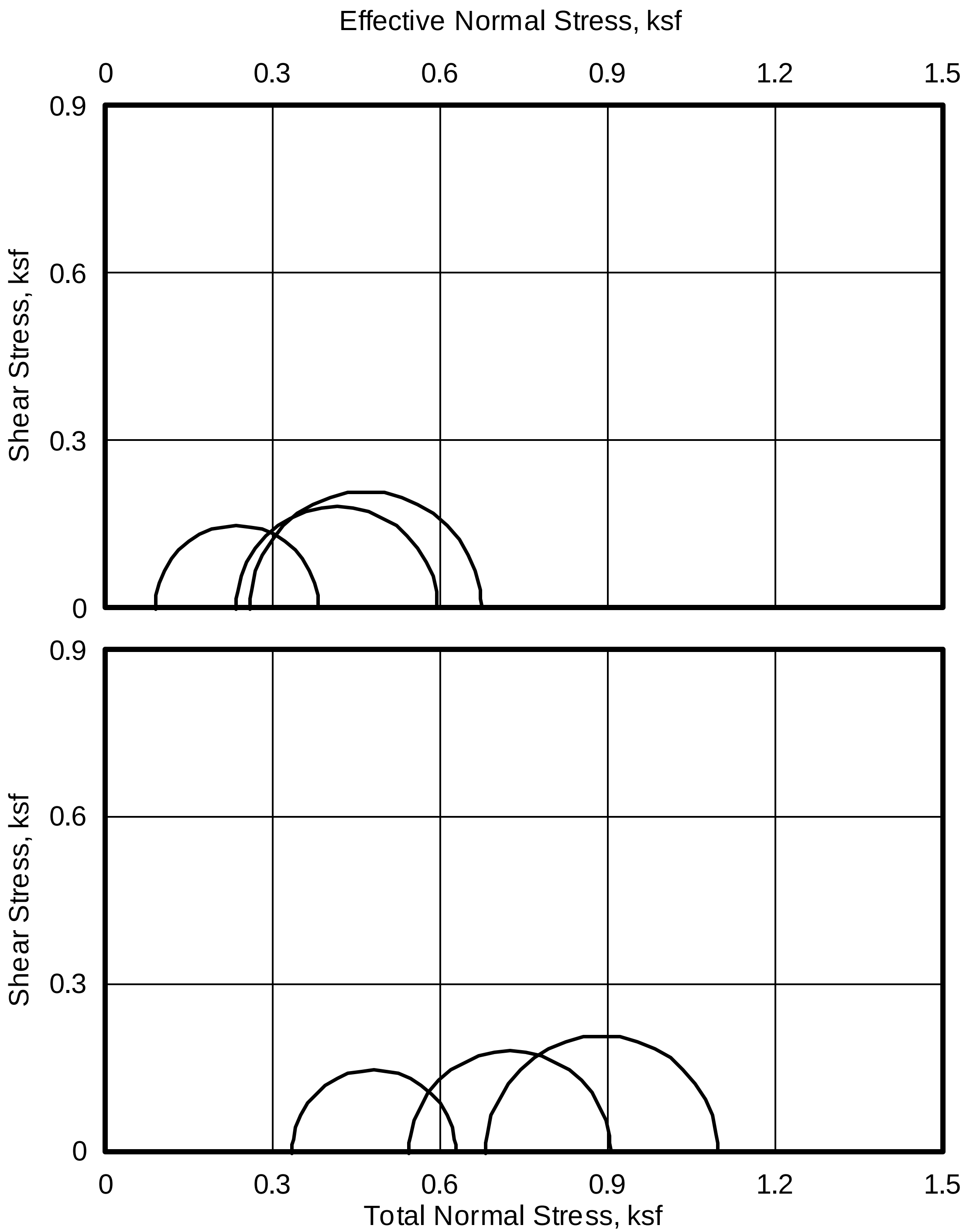
CONSOLIDATION TEST RESULTS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

BORING: TS-9
PENETRATION: 40 ft
MATERIAL: CLAY, very soft, gray
WATER CONTENT: 104.9%
INITIAL VOID RATIO: 2.89
FINAL VOID RATIO: 1.05
SPECIFIC GRAVITY: 2.7 (assumed)



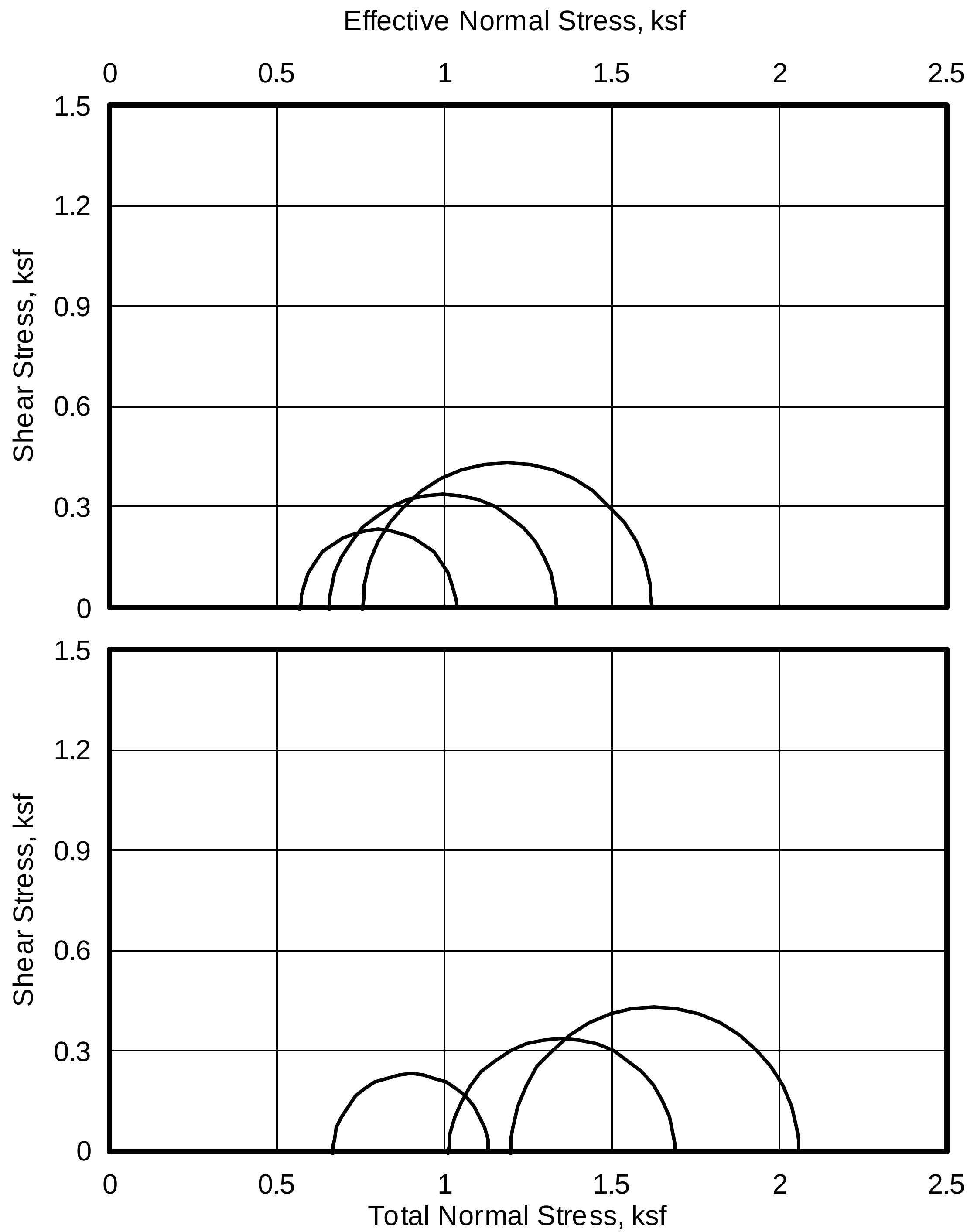
CONSOLIDATION TEST RESULTS
TEST SECTIONS – ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA

APPENDIX D



MULTI STAGE CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

BORING TS-6, 12 FT DEPTH
TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA



MULTI STAGE CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST RESULTS

BORING TS-6, 25 FT DEPTH
TEST SECTIONS - ROCKEFELLER REFUGE
GULF SHORELINE STABILIZATION PROJECT
CAMERON PARISH, LOUISIANA